THE PRINCIPLES OF IRRIGATION ENGINEERING

F. E. KANTHACK

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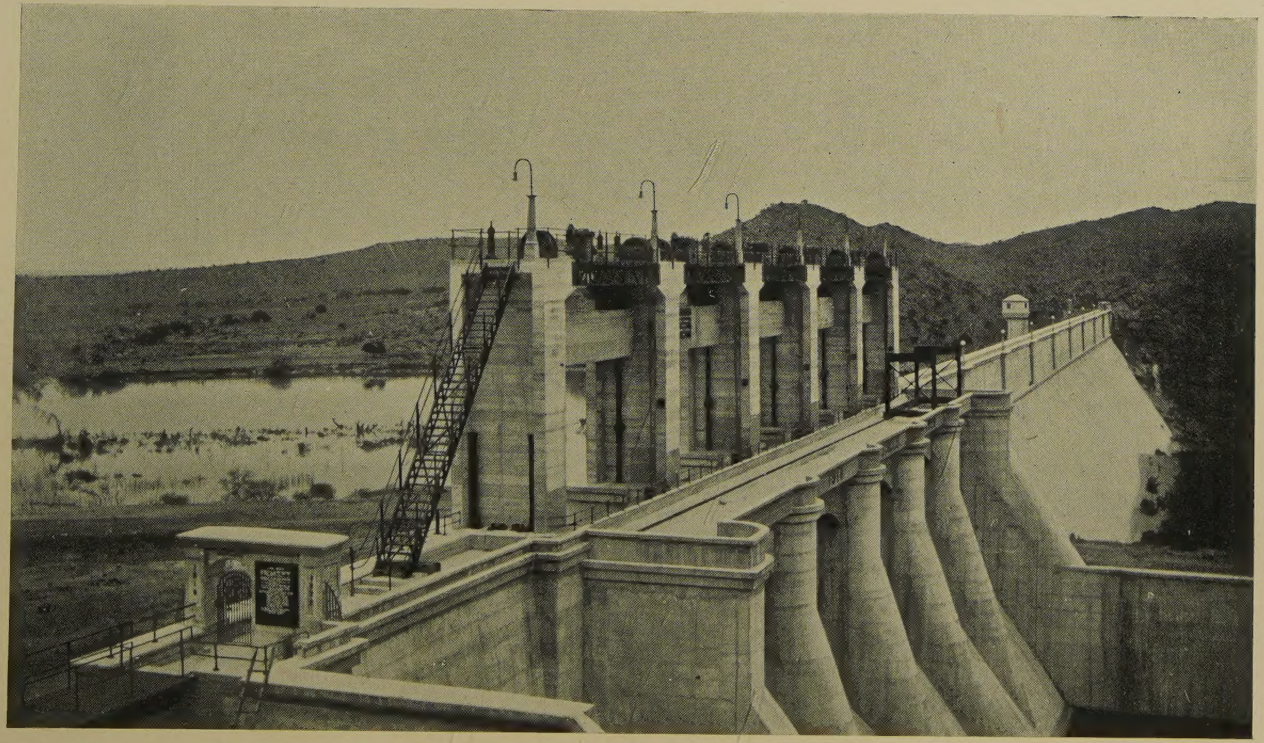

LAKE MENTZ DAM, South Africa.

THE PRINCIPLES OF IRRIGATION ENGINEERING

WITH SPECIAL REFERENCE TO SOUTH AFRICA

BASED ON A COURSE OF LECTURES DELIVERED AT THE UNIVERSITY OF CAPE TOWN

BY

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WITH ILLUSTRATIONS AND DIAGRAMS

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PREFACE

DURING 1921 the author was appointed Reader in Irrigation and Water Supply in the University of Cape Town, and delivered a short course of lectures on the subject to the engineering students. The time available for the purpose was much too short to deal in a comprehensive manner with so large a subject, and it was therefore necessary to limit the scope of the lectures to main principles underlying Irrigation Engineering and to avoid as much detail as possible.

The object of the lectures was to serve as an introduction and supplement to the numerous excellent textbooks and standard works on irrigation, mostly by American authors, which are now available. Most books of this kind are written with special reference to conditions obtaining in the United States of America, India, and various European countries, and experience has shown the necessity of bringing home to students and young engineers the differences between conditions in the Union of South Africa and those existing in other countries in which irrigation has been developed on a considerable scale. Without a proper appreciation of these differences the application of information contained in books written in other countries is very liable to lead engineers astray.

Acting on the suggestion of several professors on the staff of the University of Cape Town, the author has rewritten and enlarged the lectures in book form, but the general scope of the lectures has been adhered to.

The author has to express his indebtedness to various authors and authorities for the information of which use has been made. In the majority of cases the sources of such information have been acknowledged in the text.

Special thanks are due to Dr. Arthur Stead, Senior Research Chemist in the Agricultural Department of the Union of South Africa; Justice the Hon. J. E. R. de Villiers, now Sir Jean de

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Villiers, Judge President of the Orange Free State Division of the Supreme Court; A. D. Lewis, Director of Irrigation; Chas. Stewart, Chief Meteorologist to the Department of Irrigation; W. Van Warmelo, Hydrographic Surveyor, Union Irrigation Department; Professor A. E. Snape, Dean of the Faculty of Civil Engineering, University of Cape Town; W. Ingham, Chief Engineer, Rand Water Board; R. W. Newman, F. T. Patterson, and many other friends.

In presenting these lectures in book form the author is very conscious that his treatment of the subject is incomplete, but he hopes that it will be of assistance to students and engineers not only in South Africa but in other countries, and that it will call attention to the great limitations of our knowledge of many matters on which irrigation science and engineering is based, and will thus stimulate research.

F. E. K.

JOHANNESBURG, 1923.

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THE PRINCIPLES OF IRRIGATION ENGINEERING

CHAPTER I

INTRODUCTION

IRRIGATION is the process of artificially supplying water to the soil for the production of crops in countries where the rainfall is either deficient throughout the year or falls at a wrong season of the year to satisfy the requirements of growing crops.

Its object is essentially agricultural—the distribution of water from points where water is artificially brought in proximity to, and at levels high enough to command, the area to be irrigated being a special branch of agricultural engineering. The design and execution of the works required to supply the water in adequate volume and at the proper time is a highly specialized branch of civil engineering.

The irrigation engineer must be more than a civil engineer. He must be a civil engineer with a considerable knowledge of agricultural science, chemistry of soils, and practical irrigation farming.

A civil engineer who endeavours to design and carry out an irrigation system without having a thorough understanding of the detailed practical requirements of the farmer who is to use the water, or who is ignorant of the effect of water on the brak salts contained in many soils and of the principles of land drainage, is most certainly going to fail in his efforts.

One thing must be always kept in mind when dealing with irrigation in South Africa generally, viz., the vastness of the country and the variety of its climate.

To attempt to generalize about the agricultural conditions of South Africa is fatal. There is little or no similarity between the agricultural requirements of, say, the districts of Stellenbosch, Calvinia, Cradock, Barberton, and Pretoria, and whilst all require irrigation, the resemblance between irrigation schemes in the districts named is outward only.

Another matter of which it is essential to understand the principles is the law relating to the use of water for irrigation. Many a scheme has gone sadly wrong owing to the failure on the part of the promoter or designer to appreciate this important item.

For the growth of crops, or any vegetation for that matter, certain essentials are necessary. Firstly, soil of sufficient depth, to enable the root system of a plant to develop and obtain its necessary nourishment. The mechanical character of the soil is also important, as will be shown later. Other essentials are : water, air, and sunshine.

A soil fit for plant life must be of such a character that, apart from its containing the chemicals present in an available form necessary for the growth and development of the plant, it must be capable of retaining a considerable quantity of water and air, without which no plant can exist, let alone thrive. Given the right mechanical character to enable the soil to act as a reservoir for water and air, plants will grow on some extraordinarily barren soils. Where plants or crops are grown artificially much can be done to rectify the deficiency of certain chemical constituents required for the nourishment of the crop, by applying these to the soil in the form of natural or artificial manures or fertilizers.

Agricultural crops are very much more exacting as regards demands on the soil than the native vegetation. The latter is the result of enormously long periods of survival of the fittest and adaptation to local conditions.

This is well illustrated in South Africa, where the flora varies from place to place according to the geological and climatic conditions which obtain. Where these conditions have been uniform for a vast period of time, the adaptation of plants to such conditions, especially where these are adverse and even harsh, is very wonderful, as exemplified by the succulents of the arid regions.

Agricultural crops are not adaptable excepting within narrow limits. Each crop has definite requirements for its growth and

development. Occasionally natural conditions are such that, on the average, they suffice for all the demands made upon nature by the crop without any artificial aid other than good husbandry. This is, generally speaking, the case, for example, in the great wheat-growing countries. More often, however, some essential item or items necessary to a particular crop are lacking. It is only within certain narrow limits that such deficiencies can be made good by artificial means.

Different crops demand certain very definite basic conditions, just as the wild plants, from which they have been derived, have developed under certain definite conditions of soil and climate, and will only thrive if these conditions continue to be present. In these matters the seasonal distribution of rainfall, the absence or existence of frost, humidity, mean temperature, and other similar items play a very important roll. Subject to these basic conditions being complied with, other important requirements may, if naturally absent, be artificially applied. Such artificial supplements are in general displayed in three directions only. Firstly, the supply of water necessary to make up the deficiencies of the natural rainfall; this is effected by means of irrigation. Secondly, the removal of any excess quantity of water in the soil, as also the removal of excessive concentration of certain chemical salts in the soil by means of drainage. Thirdly, a supply of certain plant foods or chemical constituents necessary for the growth of the crop, which are naturally absent or deficient, by means of artificial fertilizing.

It will thus be seen that the successful and profitable growing of crops imposes many exacting conditions, and in opening up a tract of new country considerable experience and scientific knowledge must be brought to bear on the subject before satisfactory agricultural operations can be undertaken.

We are here only concerned with those areas where nature is met with in her sterner moods. The favoured tracts where nature has been lavish in her gifts, and it only rests with man to make reasonable use of them for the growing of rich crops, are, unfortunately, comparatively rare in South Africa.

We are concerned here with those cases, previously mentioned, where it is possible to make the soil yield a harvest useful to mankind by artificially providing those essential needs to plant life which nature has withheld. More particularly are we concerned here with the supply of an adequate amount of moisture or the removal of an excess of moisture—these being the

problems which essentially concern the irrigation engineer. As an irrigation engineer is generally also a pioneer in development, it is essential that he should have sufficient knowledge of agricultural science to discriminate clearly between a practicable scheme of irrigation and an impracticable one. Such an engineer should, for example, not be guilty of proposing to irrigate a tract of country which by reason of unsuitability of soil or inadequate depth of soil is not profitably culturable under any circumstances, or is incapable of yielding a reasonable return directly or indirectly on the capital which must be invested to bring it under irrigation.

In this connexion enthusiastic but unthinking people constantly go astray when they suggest that large areas of flat country should at once be brought under irrigation and cultivation, regardless of the fact that the depth of soil available for plant life amounts only to a few inches, and that such land is wholly unculturable in the agricultural sense, or at any rate is unfit for the cultivation of crops under irrigation.

Plants under natural conditions rely upon rainfall for their moisture. The rain, directly, has little effect upon plant life. To be effective this water must be stored for a time in the soil in which the plants are growing. If the amount of moisture in the soil is deficient or excessive, plant growth is retarded. It is through the medium of the water in the soil that the plant obtains its food, the available plant food contained in the soil being that which is capable of being dissolved and held in solution by the water.

Soil consists of particles of disintegrated rock mixed with a certain amount of organic matter.

There is a certain attraction between water and the surfaces of the rock particles, and under normal conditions there is a thin film of water enveloping each particle. This is termed hygroscopic water, and is very firmly held by the soil and has little or no direct value for the plants. The amount of such water in a soil increases as the fineness of the soil particles increases, and a soil will absorb a definite amount of hygroscopic moisture depending upon its hygroscopic coefficient.

When more water is added to the soil after its hygroscopic capacity has been satisfied, this is held more loosely by the particles, and a point is reached when the plant can begin to make use of this additional water. The percentage of moisture corresponding to the point at which the plant can just begin

to make use of the water surrounding the particles is termed the wilting coefficient. Any further moisture held by the soil in excess of the wilting point is free, and is readily available to plants. This water moves freely in conformity with the laws of capillarity.

The maximum capillary capacity is reached when the attraction between the soil surface and the water ceases to be active and the soil becomes saturated. Any water added to saturated soil will obey the laws of gravity and sink into the subsoil and drain away. This is known as "seepage." For successful crop growing the moisture content in the soil must not be allowed to fall below the wilting point. Excepting in humid regions the losses of moisture due to surface evaporation are at times greater than can be made good by occasional replenishment with rainfall. In such cases crops can be grown only if artificial measures are taken to minimize the evaporation sufficiently to retain enough moisture in the soil to satisfy the requirements of the crop, or, if this is not sufficient, irrigation must be resorted to.

The first method is known as "dry farming."

"Dry farming" consists essentially of a specialized form of tillage which, by creating a more or less impervious layer at some depth below the surface and the finest possible tilth on the surface, causes a minimum amount of moisture to be drawn to the surface by capillarity and there evaporated. It is not intended here to go into the details of "dry farming," but it is important to indicate clearly the limited conditions under which it is possible to grow crops by this means. Much misconception exists in regard to this matter, which has caused many farmers, both in this country and elsewhere, to go sadly astray.

In the first place, it is impossible to lay down a definite annual rainfall for any large tract of country above which "dry farming" methods are practicable. Under specially favoured conditions, where the rainfall happens to be regular and seasonable, and soil conditions are suitable, "dry farming" methods have been successfully applied in cases where the rainfall was as low as twelve inches per annum.

Where any or all of the following conditions obtain, viz., high mean temperature, high winds, low relative humidity, shallow or poor soils, the minimum rainfall with which "dry farming" methods can be satisfactorily applied increases considerably, and there are many places where "dry farming"

methods are impracticable with rainfall of even twenty inches. As soon as the point is reached where ordinary " dry farming " methods are insufficient to obtain sufficient conservation of water in the soil to produce uniformly profitable crops, irrigation must be employed. It is here important to note that every irrigation farmer should practise to the fullest extent possible the most approved methods of the dry farmer.

Irrigation is at its best a costly process, and the amount of water available in South Africa is, as will be shown later, very limited. The farmer should therefore do all he possibly can to conserve moisture in the soil, and to the extent he succeeds in this he will be relieved of having to supply water by irrigation.

Bearing these facts in mind it is now possible for us to consider to what extent irrigation is necessary in South Africa for the production of crops.

The geological history of South Africa is not of a kind tending to produce large areas of culturable land. When the country, which is mainly made up of high plateaux, is closely examined, the proportionately small development of culturable land is as striking as it is disconcerting.

The area of rough, hilly country formed largely by the erosion of the tableland is very great. There is still an immense area of comparatively flat country which superficially looks good, but is often found to consist of a very shallow soil overlying either the original rock or, more often, thick beds of secondary formations such as calcareous tufa or laterite. These shallow soils are sometimes unfit for cultivation of any kind and will never bear the cost of even the most simple and inexpensive irrigation works. Really good soils are confined to alluvium in the valleys of many of our rivers. Apart from these alluvial soils there are large areas of medium to poor agricultural soils all round the coastal belt and in the eastern and northern portions of the Union. Such soils are generally comparatively shallow and under present conditions are suitable only for the cultivation of maize, millets, wheat, and other cereals. Unless these soils happen to be suitable for fruit or tobacco they are rarely suited for the economical growing of crops under irrigation.

It is very difficult to estimate the total area of land in the Union of South Africa which is potentially culturable and irrigable, considering only such land which is good enough to bear the high capital charges involved in its being brought under irrigation. It is doubtful whether its total area amounts

to more than two per cent of the total extent of the country comprising the Union as at present constituted; and of this two per cent only a portion is capable of being irrigated, or suitable for that purpose, on the assumption that an adequate supply of water could be made available.

Thus out of a total area of 473,000 square miles comprising the Union, only about 10,000 square miles, or say 6,500,000 acres, can be classed as really high-class culturable land, fit to carry the high capital charges which unfortunately are inseparable from irrigation in this country. Of this area only a certain proportion is potentially and practicably irrigable.

When estimating the approximate area in the Union which is potentially irrigable regard must be had to a number of agricultural considerations. Certain land may be suitable for growing low-grade crops, such as cereals, without irrigation, whereas if high-grade crops, such as lucerne, tobacco, fruit, trees, etc., are to be grown on the same land, irrigation may be essential during the whole or during a portion of the growing season.

In the first place we have in the Union two areas with extreme conditions. Firstly, the area in which climatic conditions are such that irrigation is unnecessary for the majority of crops. This zone is very limited in area and chiefly confined to portions of the coastal belt in the east and south, and to the south-western extremity of the country. Secondly, the area in which no crops of any kind can be grown without irrigation. This zone includes the greater part of the Karroo in the Cape and south-western Free State, the whole of the arid region in the north-west and north of the Cape Province. There remains the balance of the country in which irrigation is necessary for certain crops during portions of the year, and occasionally for all crops. This zone includes most of the south-western and easterly portions of the Cape, the northern and eastern parts of the Orange Free State, and the greater part of the Transvaal and inland parts of Natal.

The necessity or otherwise for irrigation for various crops in different parts of the Union, apart from soil conditions, depends upon the total annual rainfall which can be relied upon, and more especially upon the seasonal distribution of the rainfall.

The mean annual rainfall over a district is, however, no guide to its crop-producing capacity. A heavy annual rainfall is of little use unless it can be relied on during certain critical periods of growth, and we have the common apparent anomaly, both

in this country and elsewhere, of areas with a low rainfall being able to produce, regularly, fair crops without irrigation; whereas in districts with double and treble the rainfall irrigation is essential. In South Africa the critical months of the year are August, September, and October. In parts of the Cape and in the southern Orange Free State, where winter conditions last longer, September and October are the critical months. Farther north spring conditions appear earlier, and August and September are the critical months. In these spring months crops of all kinds require water. Winter crops, such as cereals, will not mature without it, and summer crops can not be started without it.

Most unfortunately these are just the months during which there is normally little or no rain over the greater part of South Africa.

Tables I and III show the mean annual and the mean monthly distribution of rainfall over the Union, and from the latter it will be seen that the percentage areas over which the mean rainfall is below one inch during August, September, and October are 81·5, 62·3, and 33·8 respectively.

Table II gives the distribution of the mean annual rainfall over the Union and the Mandated Territory of the South-West Protectorate (formerly German South-West Africa) combined, the total area being thus increased to 800,000 square miles.

TABLE I

Mean Annual Distribution of Rainfall in the Union of South Africa, based on the Map prepared by the Meteorological Department and Published in the "Official Year Book."

Mean Annual Rainfall in Inches	Area in Square Miles	Percentage of Total Area
Under 5 ..	8,987	1·9
5·1 to 15 ..	173,118	36·6
15·1 to 25 ..	154,671	32·7
25·1 to 35 ..	108,317	22·9
35·1 to 50 ..	26,015	5·5
50·1 to over 70	1,892	0·4
TOTAL ..	473,000	100·0

TABLE II

Mean Annual Distribution of Rainfall in the Union of South Africa and the Mandated Territory of the South-West Africa Protectorate, based on Dr. J. R. Sutton's Maps accompanying his Communication to the Royal Society of South Africa (Trans., Vol. IX, Part 4, 1921).

Mean Annual Rainfall in Inches	Area in Square Miles	Percentage of Total Area
Under 5 ..	100,000	12·5
5 to 10 ..	160,000	20·0
10 to 20 ..	250,000	31·25
20 to 30 ..	180,000	22·5
30 to 40 ..	94,000	11·75
40 to 50 ..	14,000	1·75
50 to over 60 ..	2,000	0·25
TOTAL ..	800,000	100·00

From this table it will be seen to what an enormous extent the latter country is arid. More than two-thirds thereof has a mean rainfall of less than ten inches per annum, and only a very small area in the extreme north-east enjoys a mean rainfall exceeding twenty inches per annum.

Statistics of mean rainfall, whether monthly or annual, are very misleading, and to a large extent useless as a guide to agricultural requirements in the way of irrigation. The complete rainfall record must be examined in every case to ascertain the true position in this respect for a given locality.

This can best be illustrated by a few examples, which are representative of some of the main irrigation areas of the Union. A statement—Table III—showing the details of the records for the critical months of August, September, October, and November for seven stations has been compiled. The basis of the analysis is the total rainfall during each month. The length of the records of the stations selected range from thirty-five to forty-nine years in the case of six of them. Unfortunately the Transvaal records are all short ones, and that for Klerksdorp, which is the spot selected for the South-Western Transvaal, extends over twenty-five years only.

For each month the individual totals have been classified into groups as follows: *Nil*, 0 to 0·25, 0·26 to 0·5, 0·51 to 0·75, 0·76 to 1·0, 1·01 to 2·0, and over 2·0 inches; and the number of months in each class is presented as a percentage of the total year's record.

TABLE III

Showing Classification of the Monthly Rainfall Totals of Certain Stations expressed as Percentages of the Total Numbers of Years over which the Records Exist.

Name of Station	Length of Record in Years	Mean Rainfall for month	*Nil*	0 to 0·25
		August		
Bedford C. ..	43	0·75	0	18·6
Queenstown C. ..	49	0·57	8·16	36·7
Graaff Reinet C. ..	40	0·52	15·0	27·5
Beaufort West C. ..	43	0·33	16·3	35·0
Bethulie O.F.S. ..	35	0·46	34·3	14·3
Kroonstad O.F.S. ..	41	0·50	24·5	29·3
Klerksdorp T. ..	25	0·33	44·0	32·0
		September		
Bedford	43	1·92	2·3	7·0
Queenstown	49	1·10	4·1	28·6
Graaff Reinet ..	40	1·05	5·0	40·0
Beaufort West ..	43	0·67	4·7	39·5
Bethulie	35	0·68	22·9	25·7
Kroonstad	41	0·88	17·1	19·5
Klerksdorp	25	0·62	16·7	29·2
		October		
Bedford	43	2·50	7·0	30·2
Queenstown	49	1·64	0·0	4·1
Graaff Reinet ..	40	1·18	2·5	12·5
Beaufort West ..	43	0·66	7·0	27·9
Bethulie	35	1·16	11·4	8·6
Kroonstad	41	1·88	0·0	4·9
Klerksdorp	25	2·03	4·2	4·2
		November		
Bedford	43	2·80	0·0	0·0
Queenstown	49	2·13	0·0	12·2
Graaff Reinet ..	40	1·20	2·5	12·5
Beaufort West ..	43	0·71	11·6	32·6
Bethulie	35	1·23	11·4	11·4
Kroonstad	41	3·12	0·0	2·4
Klerksdorp	25	2·65	0·0	4·2

TABLE III—*continued*

Name of Station	0·26 to 0·5	0·51 to 0·75	0·76 to 1·0	1·01 to 2·0	Over 2·0
			August		
Bedford C.	13·9	9·3	18·6	27·9	11·6
Queenstown C. ..	24·5	8·2	6·1	8·2	8·2
Graaff Reinet C. ..	22·5	12·5	7·5	10·0	5·0
Beaufort West C. ..	30·2	11·6	0·0	7·0	0·0
Bethulie O.F.S. ..	17·1	5·7	11·4	11·4	5·7
Kroonstad O.F.S. ..	7·3	9·75	9·75	17·1	2·4
Klerksdorp T. ..	8·0	0·0	4·0	8·0	0·0
			September		
Bedford	7·0	7·0	9·3	30·2	37·4
Queenstown	12·2	6·1	10·2	20·4	18·4
Graaff Reinet ..	7·5	7·5	7·5	17·5	15·0
Beaufort West ..	23·25	4·65	6·9	16·3	4·65
Bethulie	11·4	2·85	8·6	20·0	8·6
Kroonstad	9·75	9·75	9·75	21·9	12·2
Klerksdorp	12·5	8·3	4·2	25·0	4·2
			October		
Bedford	14·0	18·7	2·3	20·9	7·0
Queenstown	12·2	18·4	4·1	32·7	28·5
Graaff Reinet ..	17·5	10·0	12·5	25·0	20·0
Beaufort West ..	16·3	18·6	2·3	20·9	7·0
Bethulie	8·6	11·4	8·5	28·5	22·8
Kroonstad	4·9	12·2	9·7	21·9	46·4
Klerksdorp	8·3	0·0	20·8	25·0	37·5
			November		
Bedford	2·3	4·6	7·0	27·8	58·2
Queenstown	2·0	12·3	2·0	20·4	51·0
Graaff Reinet ..	15·0	10·0	15·0	30·0	15·0
Beaufort West ..	16·3	7·0	4·6	16·3	11·6
Bethulie	8·6	11·4	2·9	40·0	14·3
Kroonstad	2·5	2·4	2·5	17·1	73·2
Klerksdorp	0·0	8·3	4·1	16·6	66·6

NOTE.—The annual means of the stations in Table III are as follow :—

Bedford	27·87 inches.
Queenstown	23·02 ,,
Graaff Reinet	13·88 ,,
Beaufort West	9·61 ,,
Bethulie	18·46 ,,
Kroonstad	23·93 ,,
Klerksdorp	21·89 ,,

Two specific cases will be described in detail. Queenstown, with a continuous record since 1873, has a mean rainfall of

twenty-three inches, the extremes hitherto recorded being 43·16 in 1891 and 12·10 in 1919.

The mean September rainfall is 1·10 inches. Now if this amount could always be relied upon during this month, and fell in one or two substantial showers, irrigation might to a large extent be unnecessary. A study of the analysis shows, however, that September is a very precarious month indeed as regards rain. In six years out of every ten the total September fall is less than one inch, and in five years out of every ten the total fall for the month is less than half an inch. Moreover, the individual falls are small.

The mean for the month is inflated by a few abnormally heavy falls, as 6·11 inches in 1913 and 4·10 inches in 1905. If these two years are eliminated the mean for the month would be reduced to 0·87 inches, or by twenty per cent.

An examination of the rainfall record will show at once that not only are irrigation facilities essential in the Queenstown area during September but also during October. Taking Bethulie as an example, the monthly means for August and September are clearly inadequate. October has a mean fall of 1·16 inches. It will be seen, however, that for nearly fifty per cent of the record the rainfall during that month was less than one inch, and for nearly thirty per cent of the period the fall was less than half an inch. In this case irrigation is essential in all the four critical months.

In deciding what provisions must be made for water in any particular locality, the monthly mean for the four critical months should in the first place be examined. If the mean is clearly inadequate to satisfy the crop requirements, then it is obvious that the land must be irrigated and further analysis is unnecessary. Such analysis becomes necessary only when the mean for the month is apparently sufficient. As previously stated, all crops require moisture during the period from the beginning of August to the end of October. From the 1st of November to the end of the irrigation season about the end of April the moisture requirements vary considerably according to the particular crops which are being grown. This aspect of the case will be dealt with at a later stage.

To better illustrate the distribution of rainfall over South Africa maps are attached showing, firstly, the mean monthly rainfall, and secondly, the mean annual rainfall. The monthly isohyets are taken from Dr. J. R. Sutton's maps accompanying

his paper on "The Study of the Rainfall Map of South Africa," contributed to the Royal Society of South Africa (Trans., Vol. IX, Part 4, 1921). The information contained in these maps is also given in Table II.

The isohyets for the annual mean are taken from the map published in the "Official Year Book of South Africa." The information embodied in this map is also given in Table I.

For a true estimation of the agricultural moisture requirements of the country more reliable data can be obtained by a careful study of the indigenous vegetation. This invariably reflects the more adverse rainfall conditions obtaining, and is never influenced by better climatic conditions which may occur at irregular intervals. On the other hand, such vegetation does not necessarily reflect rare periods of particularly severe drought. The mean rainfall which might be deduced from a carefully prepared map showing the regional distribution of vegetation would certainly be considerably lower in a great many cases than the arithmetic mean as calculated from the meteorological records. Such a map is given in the "Official Year Book of South Africa," but it is a very rough approximation. As the work of the botanical survey proceeds this map will become increasingly valuable. A moderate knowledge of the indigenous vegetation of the country in relation to climate is, however, often the truest guide to the water requirements of crops in any particular area.

From a careful analysis of the rainfall and of general climatic conditions obtaining all over the Union an approximate estimate has been prepared, showing the area during each month of the year in which irrigation facilities must be available for the proper growth of all higher-grade crops. These crops exclude maize, millets, and in many cases wheat, barley, oats, grasses, which can be grown in various parts of the Union without irrigation, and which can, as a matter of fact, rarely be grown profitably under irrigation. These areas are shown on the charts of monthly mean rainfall, and are summarized also in the last column of Table III. From this table, and from the maps, it will be seen to what an enormous extent South Africa is dependent upon irrigation for the production of high-grade crops, such as lucerne, citrus fruits, deciduous fruits, sub-tropical vegetables and fruits, vines, etc. In many cases also the output of sugar cane, cotton, and other crops now grown

largely without irrigation could be greatly increased if irrigation facilities were available.

TABLE IV

Mean Monthly Distribution of Rainfall in the Union and Areas requiring Irrigation during each Month expressed in Terms of Percentages of Total Area.

Month	Below 1 inch	1 inch to 2 inches	2 inches to 3 inches	3 inches and over	Area Requiring Irrigation
January ..	22·4	15·2	9·1	53·3	70
February ..	21·7	19·0	8·5	50·8	68
March ..	9·1	19·6	17·8	53·5	73
April ..	28·5	45·6	18·3	7·6	82
May ..	62·0	32·3	5·7	0·0	*Nil*
June ..	87·9	12·1	0·0	0·0	6
July.. ..	92·0	4·9	2·5	0·6	*Nil*
August ..	81·5	14·2	4·3	0·0	17
September ..	62·3	31·6	5·3	0·8	90
October ..	33·8	38·0	17·7	10·5	90
November ..	34·2	16·0	11·8	38·0	71
December ..	31·9	12·9	10·6	44·6	38

Irrigation may be effected in a great variety of ways, but in the majority of cases the choice of method is more or less forced upon the promoter.

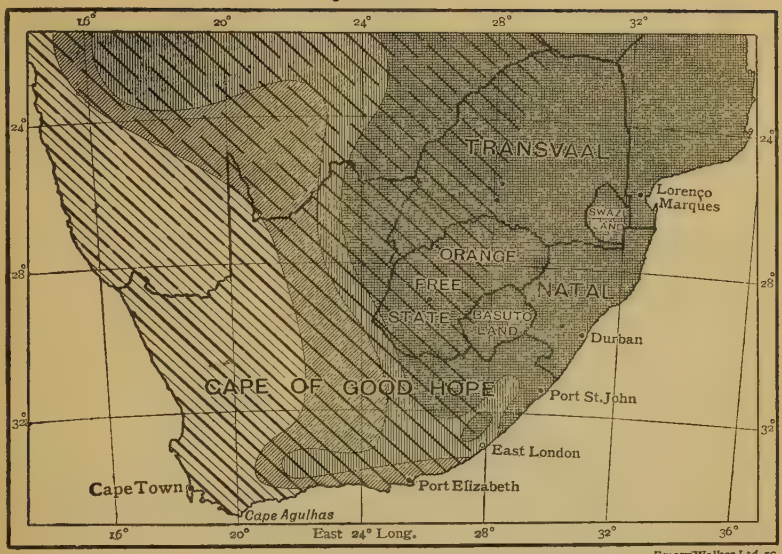
The very fact that irrigation is required implies that there is a deficiency of moisture in the area to be served; and it becomes necessary either to draw upon supplies derived from other areas, or upon reserves within the district which have been accumulated above or below the surface during more favourable seasons of the year, or during exceptionally wet years.

In India and in the East generally vast areas are irrigated regularly and efficiently with water raised from shallow wells, frequently by means of very primitive water lifts. In Egypt and the East much land is also watered by lifting water from rivers on to the adjoining land by means of similar primitive devices.

MAPS

Showing average monthly rainfall over South Africa and areas in which irrigation facilities are required. [FIG. 1 (A) to (L).]

JANUARY



Emery Walker Ltd. sc.

FEBRUARY

Emery Walker Ltd. sc.

0 to 1" | 1" to 2" | 2" to 3" | Over 3"

Area in which irrigation facilities are required

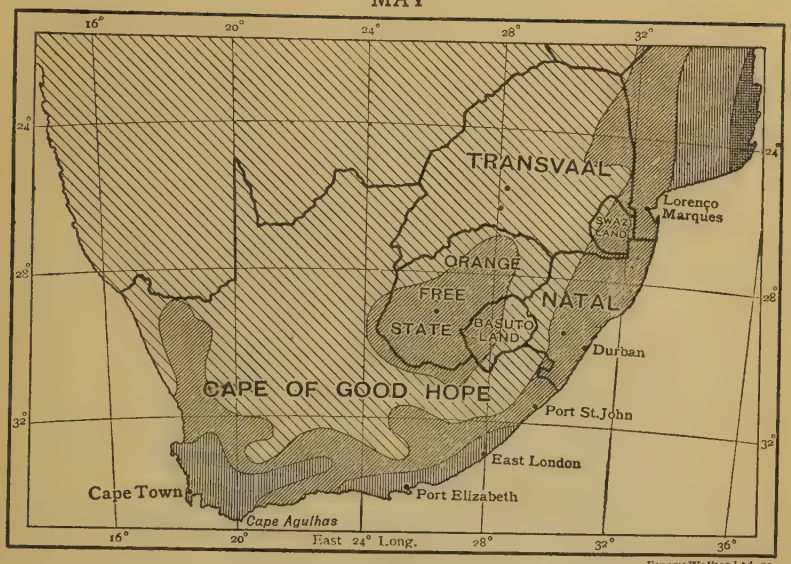
MARCH

Emery Walker Ltd. sc.

APRIL

Emery Walker Ltd. sc.

MAY



Emery Walker Ltd. sc.

JUNE

Emery Walker Ltd. sc

0 to 1″ | 1″ to 2″ | 2″ to 3″ | Over 3″ | *Area to which irrigation facilities are required*

JULY

Emery Walker Ltd. sc.

AUGUST

Emery Walker Ltd. sc.

0 to 1" *1" to 2"* *2" to 3"* *Over 3"*

Area in which irrigation facilities are required

SEPTEMBER

Emery Walker Ltd. sc.

OCTOBER

Emery Walker Ltd. sc.

0 to 1" 1"to 2" 2"to 3" Over 3"

Area in which irrigation facilities are required

NOVEMBER

DECEMBER

UNION OF SOUTH AFRICA
Average Annual Rainfall
51·1" to +70"
35·1" to 50"
25·1" to 35"
15·1" to 25"
5·1" to 15"
Under 5"
Emery Walker Ltd. sc.
Longitude East 24° of Greenwich
TRANSVAAL
ORANGE FREE STATE
NATAL
BASUTO LAND
SWAZI LAND
CAPE OF GOOD HOPE
Palapye
Louis Trichardt
Pietersburg
Leydsdorp
Woodbush
Pilgrims Rest
Komati Poort
Lorenço Marques
Nylstroom
Lydenburg
Pretoria
Belfast
Carlinton
Ermelo
Wakkerstroom
Vryheid
Utrecht
Standerton
Piet Retief
Dundee
Ladysmith
Estcourt
Pietermaritzburg
Durban
Port St. John
Kroonstad
Lindley
Harrismith
Bethlehem
Middelburg
Belfast
Roodepoort
Johannesburg
Heidelberg
Vereeniging
Rustenburg
Zeerust
Mafeking
Boshofstroom
Klerksdorp
Vryburg
Wolmaransstad
Lady Grant
Bloemfontein
Maseru
Thaba Nchu
Ficksburg
Aliwal North
Kokstad
Umtata
East London
Queenstown
Alexandria
Graaff-Reinet
Bedford
Port Elizabeth
Cape St. Francis
Kimberley
Griquatown
Hopetown
Philipstown
Britstown
Hanover
Victoria West
Wagenaars
Murraysburg
Kraai
Beaufort West
Prieska
Van Wijks Vlei
Carnarvon
Fraserburg
Sutherland
Calvinia
Clanwilliam
Van Rynsdorp
Ookiep
Caries
Pniquetberg
Ceres
Worcester
Uniondale
Amalienstein
Swellendam
Cape Agulhas
Knysna
Cape St. Blaize
Cape Town
Wynberg
Simon's Bay
Port Nolloth
Warmbad
Kuruman
Upington
Kakamas
Kenhardt

The most satisfactory method of irrigation is to divert water from a perennial river on to the adjoining lands by means of canals. The great irrigation systems of Upper India are of this type, and here is developed the greatest, most magnificent and efficient system of irrigation in the world. The sources of supply in these cases are the glaciers and eternal snows of the Himalayas.

Some fine examples of perennial schemes are also to be found in the United States of America, Egypt, Italy, Spain, and elsewhere, and in time we may hope to see the great ancient system of Mesopotamia restored on an improved and enlarged scale.

So far as perennial irrigation systems are concerned, we have made comparatively little progress since the ancient people of the East developed the science and the art. No irrigation engineer can fail to be impressed, for example, by the skill and science of the Moors, as revealed in the large irrigation works and systems of southern and eastern Spain.

Perennial irrigation systems are unfortunately impossible in South Africa, excepting on a minute scale, and less straightforward methods are required.

Unless rivers traversing hot and dry countries derive their main supplies from great lakes or snowy mountains they are subject to enormous ranges of flow. Countries in which irrigation is necessary generally have more or less defined rainy and dry seasons, and in the absence of the natural storages referred to above the river systems are alternately in high flood or practically dry. Under these conditions irrigation on a large scale is possible only if the flood waters are conserved to the fullest extent possible, and the stored water is then drawn upon during seasons of drought.

Up to a certain point intermittent irrigation by means of flood water is possible and effective.

In the Cape Province practically the whole of the irrigation up to about 1915 was due to the development of intermittent flood irrigation systems.

In very dry regions, where the lie of the country is suitable, a particular form of flood irrigation, known as basin or "zaai dam" irrigation, is the only practicable and efficient system. It is practised on a considerable scale in the districts of Calvinia, Fraserburg, and Kenhardt. Speaking generally, however, flood irrigation should be looked upon as the first or pioneer phase. Its main merit is that it is cheap. The system should be followed up as soon as possible by the construction of con-

servation works. This will have the effect of converting the system from an intermittent flood scheme to something approximating a perennial scheme of very much greater value and of far greater extent.

In South Africa, with a vast expanse of country, a very small percentage is fit for cultivation, even if water were available. Unfortunately, even with the most elaborate and efficient system of conservation and irrigation imaginable, there is insufficient water to irrigate all the land available. Seeing that water on or under the surface is the very life blood of the country, it becomes our sacred duty to devise means of making the utmost possible use of all the water falling over the country, and this is essentially the task of the irrigation engineer.

The engineer soon discovers that he is up against not one but dozens of very difficult problems when he tackles this task, and many of these problems are as yet unsolved, and to a large extent imperfectly understood.

There is no branch of civil engineering which offers more scope for research on a big scale.

In the following chapters several of these problems will be dealt with.

CHAPTER II

RAINFALL IN THE UNION OF SOUTH AFRICA

Looked at from every point of view, both the science of irrigation and the scope of irrigation are based ultimately upon the annual rainfall over a large extent of country, including the area to be dealt with.

In the introductory chapter it has been shown that the degree to which irrigation must be practised in different parts of the Union depends almost wholly upon the extent to which the rainfall in the particular area must be supplemented by artificial means in order to ensure the growth and maturing of all or certain crops. Later on, when we come to consider the practical problems of irrigation engineering, it will be seen that in the majority of cases we become involved in a complex study of the rainfall conditions over areas which are frequently of vast extent.

It is obvious, therefore, that every irrigation engineer must have a clear idea of the factors which determine the climatic conditions of the country in which he is working, and more especially of the factors which regulate the precipitation as regards quantity, type of precipitation, and seasonal distribution.

Without a proper appreciation of the determining cause of the climate of South Africa an intelligent understanding of irrigation and other hydraulic problems is impossible.

It is not intended here to enter into a discussion of the general physical causes of rain and other forms of precipitation ; this information can be obtained out of any textbook on heat or meteorology. This discussion will be confined in the first instance to a statement of the particular set of conditions which determine the amount and variation of the rainfall over South Africa, and its seasonal distribution. This preliminary statement of the case is based upon a published summary written

by Mr. Charles Stewart, Chief Meteorologist for the Union, for the " Official Year Book of South Africa."

There is a very great variation in rainfall in different parts of the Union, both as regards amount and seasonal distribution.

This distribution is dependent upon the movement and action of the permanent anticyclones, which lie off the west coast of the Cape Province, and off the east coast between the Cape Province and Australia. These high-pressure zones migrate northwards and southwards with the sun, and in addition the centres of these zones move laterally from month to month.

During April and May that to the east of the Cape Province moves westwards towards the African coast, while that on the west coast moves eastwards. At the same time an important secondary centre, or core, appears over the land, where barometric pressure increases until June or July. Between the more intense regions of this anticyclonic belt there appears to be continuous activity, which leaves a weakness near the west coast, permitting the extension northwards of inverted V-shaped depressions connected with the system of moving anticyclones to the south.

The westerly winds associated with the rear of these depressions are the rain-bearers for the west and south-west coastal districts of the Cape Province, where over seventy-five per cent of the annual rainfall occurs during the winter. This region of winter rains is, however, not extensive. Originating in the cold parts of the Atlantic, the capacity of these westerly winds for moisture is small, and after condensation, induced by the high mountain barriers which form the western boundary of the great inland plateau, they soon cease to act as rain-bearers.

During September and October the zone of high pressure disappears off the land area, merging into the South Indian anticyclone, which then returns eastwards to its summer position just off the west coast of Australia, and into the South Atlantic anticyclone, which lies a short distance from the west coast of the Cape Province. The north-easterly and easterly winds associated with the former bring in the moisture which falls over the greater part of the Union during the summer months. These winds when leaving the Indian Ocean are warm, and their capacity for holding moisture is great, and although they deposit a considerable amount of moisture in ascending on to the great plateau (evidence of which is to be seen in the high rainfall and mist belts of the more easterly parts of the Union), and thereby

decrease their absolute humidity, they still penetrate the interior with a comparatively high relative humidity.

Precipitation in the interior takes place mostly through the agency of thunderstorms, the occurrence of which is facilitated by a steep temperature gradient from about the 3,500 feet contour. Such storms are also frequent along the eastern escarpment, the resulting rainfall being often very heavy.

It may be explained here that in the southern hemisphere a "cyclone" is an area of low pressure associated mainly with strong winds revolving in a clockwise direction round a low-pressure centre.

An anticyclone in the same region is a high-pressure area, with a gentle circulation of air in a counter-clockwise direction round a centre of high pressure. In the northern hemisphere the direction of these currents is reversed.

It will be seen from the brief summary which has been quoted that the rainfall and general climate of South Africa are due to world conditions the origins of which lie altogether outside the continent.

Another factor which exercises a very pronounced influence on the climate of the country is the distribution of warm and cold ocean currents which move along the coasts of South Africa. The south-east and south coasts are washed by the warm Mozambique current coming from the Indian Ocean, whereas the west coast, from Cape Town to Benguela, on the other hand, is washed by an extremely cold current flowing from the Antarctic Ocean towards the Equator. The difference in temperature between these two currents, which meet in the region of the south-west corner of Africa, is considerable.

These conditions have operated for countless ages, and have left an unmistakable record of the comparative permanency of the climate in the various parts of South Africa as far as it has been known to us. This record or evidence is partly of a physiographical and geological nature and partly botanical. The Karroo flora of arid South Africa is of a highly specialized type, with all its parts, such as leaves, stems, and roots, made either to store water or prevent the evaporation of water absorbed. This particular type of flora will not thrive in a more humid climate, and is particularly difficult to acclimatize outside its natural home. Its evolution must have occupied a vast period of time, and implies comparative uniformity of climate, extending over an equally long era.

Whilst the climate may thus be considered more or less stable over different parts of the Union, this must not be understood to imply that the rainfall is in any sense regular from year to year.

Climate and its effect upon the physiographical peculiarities and upon the vegetation peculiar to the country does not depend upon the mean rainfall over the area, but rather upon the more unfavourable conditions which obtain.

The same mean rainfall may produce two totally different types of vegetation. In one case, where wide fluctuations in the annual rainfall are the rule, the country may be semi-arid with a corresponding type of vegetation. In another case the rainfall may be more regular, and consequently much more fertile in appearance. Thus in South Africa a mean rainfall of twelve inches may in one case give rise to a grass-covered country, and in the other case to semi-arid conditions and Karroo vegetation.

The nature of the climate, and more especially the rainfall which prevails in any given area, may be fairly accurately deduced from the wild vegetation and from its general physical appearance.

In order, therefore, to study the characteristics of a particular tract of country it is necessary to examine carefully the rainfall conditions obtaining. When dealing with irrigation problems, apart from the degree to which irrigation is necessary for the growing of crops (this being determined by a study of climatic and soil conditions generally), we are in the first instance concerned with the water supply available. Whether it is a question of irrigation from a perennial stream or from a reservoir the problem of estimating the amount of water available from year to year is the foundation of everything, and immediately we begin to investigate this, the hydrographic problem, we become involved in a study of the rainfall, which is the basis of this supply.

The main sources of water supply for irrigation are rivers and springs, and these, with few exceptions, which need not be dealt with here, derive their supplies from the atmosphere in the form of rain, snow, dew, etc.

Leaving out of consideration, for the present, irrigation depending upon wells, boreholes, or permanent springs, the main supplies of water for irrigation are in South Africa drawn from rivers. The water in these rivers, whether perennial or

intermittent, is due to rain falling over their catchments, or drainage areas, and running directly off the surface into the streams or percolating through the subsoil into these streams.

The fundamental problem in almost every case is the determination of the annual " run-off," forming the available stream flow, in relation to the rainfall over the catchment. As will be demonstrated in this and the next chapters this is a problem of great difficulty and generally incapable of exact solution. To arrive even at an approximation much experience and judgment are required.

The rainfall is measured at about 2,500 places in the Union and at about 150 places in the South-West Protectorate ; that is to say, there is one rain-gauge for every 200 square miles in the Union, and a gauge for about every 2,100 square miles in the Protectorate. These gauges are not evenly distributed, however. In the great desert and semi-desert tracts of the Kalahari, Bechuanaland, Bushmanland, etc., which comprise an enormous area, there are very few meteorological stations. In other parts of the country they are comparatively close together. The meteorological department of the Union has of necessity to rely almost entirely upon voluntary services, and it thus follows that the number of observing stations is directly proportional to the degree of development of the country.

The periods over which observations extend vary very much, and in this respect it is a question of historic development and progress. The oldest settled parts of the Cape provide also the longest records. The Abbé de la Caille in the year 1751 was the first to keep systematic meteorological records in South Africa. The record at the Royal Observatory in Cape Town dates from 1841, and there are several other records in the Cape Province extending over periods of about seventy years.

The systematic collection of meteorological data commenced in the Cape long before any organized efforts in this direction were made elsewhere in South Africa, and we are fortunate in possessing many records of forty years and more in that Province. The Transvaal records are nearly all comparatively short, most of them beginning after the Boer War. Free State and Natal records are mostly of short or moderate length. The longest record in Natal was commenced at the Botanical Gardens in Durban in 1871, although some earlier but broken observations were taken at " Gardens Cliffe," starting in 1866.

In the case of all stations reporting to the Meteorological

Department, particulars are available of the rainfall on each day, provided the fall amounts to 0·01 (one hundredth) of an inch, which constitutes a "rain day." Observers also submit general remarks about the weather, which are frequently of considerable importance. The returns are tabulated for each month, for seasonal periods, and for full years.

For each of these periods arithmetic means are calculated as soon as the length of record amounts to three years. These arithmetic means, or averages, are popularly used as a gauge of the climate for any particular area.

It has been established that if observations of rainfall at any one place are carried out over a long series of years the arithmetic mean of all the annual totals tends to become a fixed amount, which in subsequent years will have but a small variation.

When we speak of the mean rainfall of a place we generally assume that we are dealing with such a fixed figure, and it is therefore necessary to investigate the nature of this important number. In discussing this and other rainfall problems attention will have to be confined mainly to the plateau area of the Union, more particularly to the drier regions thereof in the Cape and Free State Provinces, omitting for the present the coastal belts on the eastern and southern margins of the Union. These latter areas lie to a large extent outside the irrigation zone and are of much less importance to the problems we are here dealing with.

Many years ago Mr. (later Sir Alexander) Binnie analysed the rainfall of Great Britain, and later on a number of places scattered all over the world. He confined his attention to places having an average rainfall of twenty inches and over, and within certain limits succeeded in establishing a number of general laws which have been of very great value to water engineers in helping them to estimate the rainfall of an area in respect of which no data, or only inadequate data, existed.

Binnie found that irrespective of the position of any station in regard to latitude or longitude or altitude above sea level, or the total annual average rainfall (provided it is above twenty inches per annum), the fluctuations to which the annual average is subject are practically the same in all cases. He found further that the percentage of the number of years during which the annual rainfall was above the mean was 47·10, and that below the mean was 52·90.

Also that the average fall for all the years with a rainfall below the mean annual only amounted to 0·85 of the mean, and for the years above the mean annual 1·17 above the mean. In other words in fifty-three years out of every hundred the rainfall is fifteen per cent short of the mean average fall. Binnie also drew attention to the fact that in any series of years there occur periods of three years in succession during which the fall only averages 0·75 of the mean, that is, it falls twenty-five per cent below it; and, finally, that for long periods, varying from three up to nine years in succession, the annual rainfall never rises to the mean average, only amounting to 0·82 of that amount. Binnie's laws and deductions are well known to all hydraulic engineers. The results of his investigations were presented to the Institution of Civil Engineers in 1874 and in 1892, and are summarized in his book on "Rainfall, Reservoirs, and Water Supply," published in 1913.

In order to establish laws and deductions of this kind a large number of stations with long records are necessary. Binnie's final deductions were based upon 153 stations with an aggregate record of nearly 7,000 years. These included the record at the Cape Town Observatory.

Let us now apply Binnie's methods of analysis to South African conditions.

Taking, first of all, the fluctuation from the annual average. Binnie constructed curves showing the deviations from the mean for periods of observation extending from one year to thirty-five years. He found that the greatest variations for one year were fifty-one per cent above the mean and 40 per cent below the mean. Similarly for two years 35 per cent above and 31 per cent below; for three years 27 per cent above and 25 per cent below; for five years 15 per cent above and 15 per cent below; for ten years 8·22 per cent above and 8·22 per cent below; for twenty years 3·24 per cent above and 3·24 per cent below; for thirty-five years 1·78 per cent above and 1·78 per cent below.

Similar diagrams have been prepared for a number of stations in the Cape and Orange Free State, and the results obtained are of a very different order. It is found, approximately, that the smaller the annual mean the greater are the deviations therefrom for short periods.

Thus the curve for Bedford, which has a mean rainfall of 28·87 inches and is just within the rainfall area influenced by coastal conditions, approximates very closely to Binnie's curve.

TABLE V

Extreme Percentage Variations of Rainfall from Average for Various Periods of Years

Name of Station	Length of Rainfall Record in Years	Average Rainfall in Inches	Length of Period in Years—Above Average									
			1	2	3	4	5	10	15	20	30	40
			Extreme Percentage Variations from Mean									
1. Van Wijks Vlei	36	6·57	118·7	55·3	45·7	32·4	24·8	14·6	9·6	7·0	2·4	—
2. Kamfer's Kraal	45	8·62	163·3	79·1	43·5	33·6	35·4	18·4	11·8	5·1	3·5	2·6
3. Rietfontein (Willowmere) ..	48	9·37	135·1	126·7	88·9	71·6	66·3	47·1	28·3	23·0	13·7	5·4
4. Oudtshoorn	42	9·42	75·4	45·0	30·4	28·7	25·2	11·4	7·0	5·9	2·1	0·7
5. Brakfontein	48	9·91	76·3	54·7	42·1	30·5	25·0	18·7	14·0	7·1	5·8	3·7
6. Jansenville	43	10·74	66·6	54·6	38·3	38·7	30·2	23·9	13·7	8·7	6·2	2·4
7. Aberdeen	42	12·28	57·3	51·0	40·0	30·4	28·5	14·7	10·6	5·9	4·0	0·7
8. Worcester	45	12·33	97·2	74·7	68·0	63·7	46·7	11·8	7·2	4·7	2·8	1·2
9. Robertson	43	12·54	46·6	39·2	39·6	30·3	26·6	15·0	12·2	7·0	4·0	0·6
10. Graaff Reinet	40	13·88	48·5	36·2	35·3	30·2	26·2	21·7	11·2	6·8	4·4	0·0
11. Cradock	43	14·51	82·5	61·8	52·2	44·8	38·1	23·8	14·5	10·3	4·7	2·8
12. Kimberley	36	16·08	87·6	59·1	39·2	29·7	25·3	13·6	11·0	9·8	3·0	—
13. Steynsburg	40	16·50	70·9	51·9	45·4	44·7	42·2	25·1	19·0	13·2	5·0	0
14. Daggaboer	33	18·48	70·8	42·5	31·8	26·6	26·1	13·2	6·6	1·9	1·6	—
15. Tarkastad	43	18·57	84·4	57·7	51·8	46·6	42·6	24·8	15·7	8·6	4·0	1·5
16. Aliwal North	38	20·63	73·4	52·0	42·6	37·8	37·3	28·1	19·1	7·7	3·2	—
17. Bloemfontein	38	21·67	60·2	46·4	48·4	43·5	38·6	22·5	13·2	8·2	1·9	—
18. Port Elizabeth	54	22·53	51·3	34·5	27·6	21·2	18·7	13·4	7·7	6·0	0·0	1·6
19. Queenstown	48	23·02	87·5	53·7	52·3	41·0	38·8	19·5	15·8	8·0	5·1	1·4
20. Kroonstad	41	23·93	54·3	34·7	26·6	24·7	23·3	15·0	11·1	7·8	2·4	0
21. Pretoria	30	27·43	89·1	38·4	20·8	22·0	21·8	15·1	5·5	1·5	0	—
22. Bedford	43	27·87	37·2	26·7	22·5	17·9	13·3	10·0	5·4	1·8	3·0	0
23. Grahamstown	43	28·13	51·2	32·1	27·1	23·6	20·7	16·1	12·6	9·0	4·3	2·3
24. Ceres	43	41·17	51·6	34·3	25·1	15·2	14·2	10·9	7·9	6·9	3·6	0·7
Binnie's Average Figures ..	—	Below 20·0	51·0	35·0	27·0	—	15·0	8·2	4·75	3·2	2·26	—

TABLE V (*continued*)

Name of Station	Length of Rainfall Record in Years	Average Rainfall in Inches	Length of Period in Years—Below Average									
			40	30	20	15	10	5	4	3	2	1
			Extreme Percentage Variations from Mean									
1. Van Wijks Vlei	36	6·57	—	0·6	4·4	4·6	10·2	23·7	32·6	29·7	58·6	76·7
2. Kamfer's Kraal	45	8·62	4·1	3·2	8·5	10·7	17·9	25·0	30·4	38·6	47·1	64·0
3. Rietfontein (Willowmere) ..	48	9·37	7·7	13·7	21·5	23·8	28·7	38·5	34·6	42·5	45·3	66·0
4. Oudtshoorn	42	9·42	0·4	3·1	7·2	10·1	16·8	22·9	25·7	30·9	39·5	46·7
5. Brakfontein	48	9·91	0·0	0·0	8·7	8·7	15·1	24·6	27·6	32·4	43·4	64·5
6. Jansenville	43	10·74	0·0	0·0	7·7	8·8	9·4	19·9	25·0	31·6	39·0	59·2
7. Aberdeen	42	12·28	0·0	1·7	6·1	6·4	7·2	20·4	20·8	30·0	44·8	63·5
8. Worcester	45	12·33	6·2	6·9	13·8	13·9	15·0	26·5	30·3	30·5	43·6	52·0
9. Robertson	43	12·54	0·2	0·9	7·3	4·3	9·6	22·3	25·1	29·1	28·8	37·2
10. Graaff Reinet	40	13·88	0·0	2·0	5·9	7·5	9·3	15·2	20·0	28·2	37·5	55·0
11. Cradock	43	14·51	0·3	2·5	9·2	9·6	12·5	20·7	19·7	25·2	43·2	71·7
12. Kimberley	36	16·08	—	0·9	4·2	7·1	16·9	19·4	18·6	22·0	33·7	50·9
13. Steynsburg	40	16·50	0	1·7	10·3	12·7	13·7	17·9	20·9	25·4	35·5	57·0
14. Daggaboer	33	18·48	—	1·0	7·1	7·3	13·7	20·9	23·3	31·7	42·2	60·8
15. Tarkastad	43	18·57	0·2	0·3	8·9	10·0	11·4	16·3	24·6	32·4	47·8	52·3
16. Aliwal North	38	20·63	—	6·0	13·4	12·9	15·6	29·4	22·5	32·5	38·8	56·8
17. Bloemfontein	38	21·67	—	0·3	9·8	9·8	13·6	19·3	21·6	22·6	32·3	46·3
18. Port Elizabeth	54	22·53	4·2	8·1	11·7	12·3	16·3	19·5	24·2	28·7	38·3	53·8
19. Queenstown	48	23·02	0	0	5·9	6·8	15·4	20·3	30·0	34·0	36·2	47·4
20. Kroonstad	41	23·93	0	3·1	8·5	8·1	12·3	15·0	21·8	18·1	22·7	41·5
21. Pretoria	30	27·43	—	0	6·2	10·2	12·3	17·8	21·3	19·7	22·8	53·0
22. Bedford	43	27·87	0	1·5	3·6	6·2	10·9	15·2	23·5	29·2	32·7	47·4
23. Grahamstown	43	28·13	0·1	3·1	5·2	4·8	11·7	20·2	17·4	26·1	30·4	36·8
24. Ceres	43	41·17	0·7	3·6	8·1	6·2	10·7	17·5	22·7	25·3	27·8	41·9
Binnie's Average Figures ..	—	Below 20·0	—	2·26	3·2	4·75	8·2	15·0	—	25·0	31·0	40·0

For Van Wijks Vlei, on the other hand, with a mean rainfall of only 6·57 inches, the deviations from the mean for one year amounted to 119 per cent above and 77 per cent below the annual mean.

Kamfer's Kraal (Beaufort West District) shows a deviation of 163 per cent above and 64 per cent below. Beaufort West 90 per cent above and 67 per cent below. Jansenville 67 per cent above and 59 per cent below. Queenstown 87·5 per cent above and 47·4 per cent below.

All the results obtained are shown in Table V, and some of the curves[1] are also shown, from which it will be seen that until a rainfall record extends over a period exceeding thirty years it is of doubtful value. A twenty-year record may vary 10 per cent above and 5 per cent below the true mean. A fifteen-year record may vary as much as 20 per cent above and 15 per cent below the mean. In all the cases investigated the records extend over periods exceeding thirty-six years, and in most cases they exceed forty years.

To illustrate the danger of working with short rainfall records of, say, five years, an example may be given. If we took the rainfall at Queenstown over the five-year period 1889 to 1893 the resulting mean would be found to be no less than 39 per cent higher than the mean calculated over a period of forty-eight years. Similarly, if the mean rainfall for Aberdeen were calculated over the five-year period 1915 to 1919, it would be found to be 20 per cent below the mean calculated over a period of forty-two years.

The first point it is desired to notice, therefore, is that where rainfall records in the drier parts of South Africa are concerned Binnie's "deviation from mean" diagram is inapplicable; and the second is that throughout the plateau regions of South Africa the mean rainfall of any particular station whose record is less than twenty years old is of very little value to the irrigation engineer; and furthermore, any mean must be accepted with considerable reserve unless calculated over a period exceeding thirty years. Means calculated over short periods of fifteen years and less are, taken by themselves, useless.

There is a popular theory that over large areas the rainfall varies in cycles, that is to say, wet and dry periods alternate in accordance with some unknown cosmic influence. The study of any long record shows that wet and dry periods, sometimes extending over a considerable period of years, do certainly

[1] For Fig. 3 see end of book.

exist. Thus in the central portions of the Cape Province the period from 1876 to 1885 was generally very dry, whereas the succeeding ten years—1886 to 1895—were generally wet years. At many stations this latter period contains the highest maximum recorded rainfall. Subsequent to 1895 there have been no such long periods either consistently wet or consistently dry. If true cycles really exist a very much longer record will be necessary to reveal their nature.

A number of diagrams have been prepared for different stations, showing for each year the percentage above or below the mean for the whole period, and some of these will illustrate at a glance the danger of working with means calculated over short periods of time.

Binnie's investigations of world-wide scattered stations with averages exceeding twenty inches showed that the percentage of the number of years during which the annual rainfall was above the mean was 47·10, and that below the mean was 52·9. Those percentages have been calculated for thirty-three South African stations, and the results are generally in accordance with those calculated by Binnie.

In the case of places with very low rainfalls the percentage number of wet years is generally smaller than Binnie's 47·1.

In some cases the position was slightly reversed, the percentage number of wet years being slightly in excess of the percentage number of dry years.

Speaking generally, however, it may be assumed that there is a greater number of years in which the rainfall is below the mean than above the mean. There is really nothing very extraordinary about this fact. At the worst the annual rainfall could only be *nil*. There is no limit of wetness, however. This fact is illustrated clearly by the " Deviation from Mean " diagrams previously referred to, the plus deviations being in all cases greater than the minus deviations. Binnie's deductions regarding extended periods during which the annual rainfall is consistently below the mean is borne out by an analysis of any South African record, and is clearly shown in the diagrams for different stations, representing the annual rainfall in terms of percentages above or below the mean.

Take, for example, the case of Aliwal North. This record extends over a period of thirty-eight years. From 1887 to 1896, inclusive, the annual rainfall was consistently above the mean, the percentage excess varying from 4 to 73. During

the eleven-year period from 1910 to 1920 the annual rainfall was consistently below the mean, the percentage deficiency varying from 0·7 to 33, and was on the average 14 per cent below the mean, which in this case is 20·63 inches.

In the case of Beaufort West, with a mean rainfall of 9·61 inches, we have an instance of nine consecutive years over which the average deficiency amounted to no less than 22 per cent. The period in question was from 1880 to 1888.

It will be seen that these deficiencies over long periods of years are considerably greater than those given by Binnie for similar periods, and if smaller periods are considered the excesses on Binnie's percentages are equally marked, more especially where places with low mean annual rainfalls are concerned.

The third point it is desired to notice, therefore, is that Binnie's assumptions regarding rainfall deficiencies over groups of three years, or over longer periods up to nine years, are inapplicable to South African conditions.

Whilst a more or less stable mean annual rainfall may be obtained from a record of, say, forty years, such a record is quite inadequate to enable us to obtain any clear idea of the variations in the rainfall which may be reported in individual or groups of years.

Frequency or probability curves for annual rainfall would be exceedingly useful, but it is obvious that, for the drier portions of the country, at any rate, a much more extended record of rainfall will be necessary before dependable frequency curves can be calculated. A probability curve would indicate the expectancy of obtaining a given annual rainfall in, say, a period of one hundred years. Such curves are particularly useful in determining the probable frequency of excessive rainfall and consequent violent floods, or of abnormally dry years.

One of the objects of Binnie's investigations was to discover in how far it was possible to extend for a particular rainfall station a comparatively short record by deductions from other stations with similar rainfall conditions and with longer records. Within certain limits he was successful in doing this. His method as applied to Central India appears to be satisfactory, but is of much more restricted application in South Africa. Indian rainfall is determined by the monsoons, which affect in a similar manner a vast area. The great plateau area of South Africa, which obtains practically the whole of its rainfall

TABLE VI

Annual Percentage Variations from Means for Certain Stations.

Year	New Bethesda	Wellwood	Graaff Reinet	Winterhoek
1886	+39·2	+45·2	+54·6	+47·3
1887	+23·6	+15·8	+14·3	+18·7
1888	− 2·4	+ 5·6	+ 4·3	+13·7
1889	+17·0	+18·5	+ 3·4	+12·8
1890	+72·8	+45·8	+24·0	+15·4
1891	+60·0	+51·4	+48·5	+46·1
1892	−18·4	+ 0·3	+14·6	+38·9
1893	+49·8	+10·7	+10·4	+20·5
1894	+ 3·6	+28·1	+ 6·8	+ 9·3
1895	+ 7·2	+19·9	−11·0	+ 8·3
1896	+23·2	+19·3	0·0	−17·1
1897	−19·1	−20·0	−12·0	−23·4
1898	−12·3	+10·2	− 2·7	−14·0
1899	−25·4	−26·2	−34·4	−19·9
1900	+61·3	+26·2	+18·2	+15·9
1901	+19·2	+20·8	+ 9·1	+ 2·9
1902	−19·3	− 5·7	+ 8·9	− 3·7
1903	−75·8	−46·8	−43·5	−33·9
1904	− 7·2	−15·2	−12·5	−16·4
1905	−14·2	−17·8	− 6·8	+ 9·7
1906	+28·9	+26·0	+29·2	+35·1
1907	−10·9	− 7·7	−12·3	−12·1
1908	−38·5	−47·2	−48·7	−49·8
1909	− 5·3	+12·6	+33·2	+ 6·6
1910	−25·8	− 3·5	− 6·2	−10·7
1911	+ 8·9	+31·2	+27·0	+15·3
1912	−56·0	−57·8	−37·7	−36·7
1913	+21·6	+13·1	+27·1	+18·7
1914	−16·5	−38·8	−10·7	−19·2
1915	−48·0	−51·0	−38·4	−41·2
1916	− 5·3	− 4·5	− 5·3	− 8·9
1917	+11·8	+24·2	+42·6	+45·8
1918	+ 9·4	+ 0·6	−20·0	−27·3
1919	−51·3	−39·0	−55·0	−55·0
1920	− 1·9	− 3·1	+27·8	−30·4

TABLE VI (*continued*)

Year	Aberdeen	Kendrew	Cranemere	Jansenville
1886	+40·0	+34·8	+62·6	+29·6
1887	−28·0	−25·2	+20·8	+24·1
1888	−15·7	−14·1	+13·4	+ 1·9
1889	+10·0	+56·3	+55·0	+21·9
1890	+53·2	− 8·8	+11·1	− 1·1
1891	+56·5	+68·4	+58·6	+61·6
1892	− 4·4	+32·5	+22·0	−25·0
1893	+16·2	+24·0	+26·6	+66·7
1894	+20·6	− 1·7	+ 1·8	+42·6
1895	− 4·2	−26·6	+18·0	+ 5·5
1896	− 3·9	+ 4·0	−12·9	+39·9
1897	− 7·4	− 7·0	−21·9	−28·8
1898	−18·7	+ 5·0	+21·5	−13·3
1899	−19·2	−39·5	−35·7	−52·9
1900	+ 4·8	+11·7	+13·5	− 4·7
1901	+ 6·9	+21·2	+22·8	− 0·4
1902	−18·7	+ 1·0	+19·3	+ 7·7
1903	−41·8	−39·6	−41·5	−37·3
1904	−29·3	− 6·0	−31·4	−10·5
1905	+24·2	+33·2	−20·5	+45·2
1906	+54·5	+66·6	+36·3	+18·7
1907	+ 6·3	+12·0	− 2·0	+ 4·6
1908	−45·4	−37·6	−27·2	−50·2
1909	+ 3·8	+17·6	− 9·2	−25·8
1910	− 7·7	−14·2	−15·1	− 8·6
1911	+25·8	+21·3	+35·4	+25·4
1912	−13·1	−21·0	−43·0	− 8·8
1913	+46·5	+35·4	+19·1	+39·4
1914	−25·0	− 4·0	− 1·0	+23·5
1915	−63·5	−48·2	−52·3	−59·0
1916	+ 1·2	− 7·0	−18·2	−18·2
1917	+36·1	+ 1·6	+15·7	+61·2
1918	−28·0	−33·4	−27·1	−25·0
1919	−48·1	−54·5	−70·2	−52·8
1920	+22·2	+11·7	0	− 2·6

from scattered thunderstorms, presents, however, a totally different problem.

In the first place, the determination of the annual rainfall at one station cannot possibly be based upon other stations, unless these are comparatively near and are known to be subject to practically identical climatic influences. Binnie's prediction of the rainfall at Nagpur from the record at Bombay, 500 miles distant, would be an impossible feat in South Africa.

Within somewhat wide limits determination of the total rainfall for a single year is possible even in the drier parts of South Africa, and occasions frequently arise when such determinations are helpful and even necessary when making estimates of run-off.

The method will be demonstrated by actual examples. The area selected is approximately 4,000 square miles in extent, and forms a considerable portion of the catchment area of the Sundays River, situated in portions of the Graaff Reinet, Aberdeen, Jansenville, and Pearston districts in the midlands of the Cape Province.

The area is wholly in the thunderstorm region of the Karroo, and the topography is rough and mountainous in the northern and north-eastern portions.

Eight rainfall recording stations, all with records extending back to 1886, have been selected as representing the area as a whole. The mean annual rainfall for these stations ranges from 10·74 inches in the case of Jansenville to 17·14 inches in the case of Winterhoek, the latter being an exceptionally rainy spot. Table VI shows for each of these eight stations, for each year from 1886 to 1920, the percentage variations from the annual mean calculated over the whole of the existing record. The relation between each year's rainfall to the mean annual can be better shown on a large-scale chart, but such a chart does not lend itself to reproduction. The most that can be said for this and other similar comparisons is that, in general, the rainfall at each place follows the same tendency. Whenever the percentage variations from the mean are appreciable they are similar in sign, though individual percentage excesses or deficiencies often vary greatly. Thus in 1889 the rainfall throughout the catchment was in excess of the mean, but the percentage excesses varied from 3·4 to 56·3. Similarly in 1917—a year of excess rainfall at all stations—the percentage excesses varied from 1·6 to 61·2. In the year 1919—a year of general

deficiency—the individual percentage shortages varied from 39 to 70.

The percentage variation may be very great in one year and very small in another, that is to say, there is no uniformity in the behaviour of different stations in different years.

If an analysis of this kind is applied to a much larger area the results are somewhat bewildering. A chart showing the variations from mean over the whole Union of South Africa for the year 1920 is published in the Annual Report of the Director of Irrigation for the year 1920 to 1921. The Chief Meteorologist, in explaining this chart, draws attention to the somewhat extraordinary juxtaposition of areas having considerable surpluses to areas having large deficits, but, in the present state of our knowledge of the subject, does not attempt to explain the reason for the numerous peculiarities revealed by the chart; and he calls attention to the difficulties that would be encountered by anyone endeavouring to frame a seasonal forecast applicable to the whole country.

The problem which the irrigation engineer generally is called upon to solve deals fortunately with relatively much smaller areas, and, on the whole, conditions are less abnormal.

TABLE VII

Illustrating Method of Calculating the True Mean Rainfall of a Place having a Short Record only from the Means of Surrounding Stations with Long Records.

Name of Station		Mean Rainfall for Period 1911 to 1920	True Mean for Full Period of Record	Ratio Cols. 3, 4	Means of 7 other Ratios	Calculated True Means	Percentage of Error
1	2	3	4	5	6	7	8
1	New Bethesda	12·21	14·00	0·870	0·920	13·27	5·2−
2	Wellwood ..	11·66	13·34	0·874	0·920	12·68	4·9−
3	Graaff Reinet	13·21	13·88	0·952	0·909	14·53	4·7+
4	Winterhoek ..	15·80	17·14	0·922	0·913	17·31	1·0+
5	Aberdeen ..	11·72	12·28	0·954	0·909	12·89	5·0+
6	Kendrew ..	10·80	12·00	0·900	0·916	11·79	1·7−
7	Cranemere ...	10·72	12·49	0·858	0·922	11·63	6·9−
8	Jansenville ..	10·56	10·74	0·983	0·904	11·68	8·9+

The first problem which frequently has to be solved is the calculation of the true annual mean rainfall for a station with only a comparatively short record—say ten years. Table VII shows the interrelationship between the eight stations in question.

Assuming that for any of these stations only a ten-year record existed, say 1911 to 1920. The third column shows the arithmetic mean as calculated for the ten-year period. Column 4 shows the actual mean in each case calculated over periods upwards of thirty-five years. In Column 5 is given the ratio of the short mean to the true mean in each case.

In the sixth column there is shown opposite each station the mean of the ratios of the remaining seven stations, and the calculated true means shown in Column 7 are obtained by dividing the short mean in Column 3 by the mean ratio for the other seven stations given in Column 6. In the last column are shown the percentages by which the calculated true mean is wrong in each case. These percentage errors show that with the exception of Jansenville the calculated means are reasonably accurate and fall within the range of deviation from the true mean which might be expected from a record of thirty years, as shown by Table V.

The relatively high percentage of error in the case of Jansenville is what might be expected from a station with a low and erratic rainfall, and it is doubtful whether calculations of the kind here dealt with can be applied with safety to places bearing a mean annual rainfall, of less than ten inches.

If we now apply the above method to other stations in the catchment, but not included in the list shown in Table VI, a slight modification of the procedure described above is advisable. Supposing it is desired to find the true mean annual rainfall for Toegedacht. This place is approximately twenty-one miles S.E. of Kendrew, fourteen miles N. of Jansenville, and twenty-four miles S.E. of Cranemere. In this case the desired mean should be calculated on the basis of the average ratios of the three stations named, as the climatic and topographical conditions of these four stations are more alike than those of the other stations.

The mean calculated by this method is 9·80 inches, the mean for the ten-year period 1911 to 1920 being taken as a basis. A twenty-six-years' record for Toegedacht actually exists, and the resulting mean annual rainfall is 9·31, but as the long wet period 1886 to 1894 is not included, the calculated mean is obviously more accurate than the mean for the twenty-six years 1895 to 1920.

Similarly in the case of Roodebloem, which is located due north of Graaff Reinet, the calculation has been based upon the average of the ratios for the four stations New Bethesda, Wellwood, Graaff Reinet, and Winterhoek. The calculated mean in this case is 11·06 inches compared with an actual mean of 11·60 calculated over a period of thirty-five years. The percentage of error in this case is less than five.

The second problem to be solved is the calculating of a single missing record or the extension backwards of a short record. The two cases of Toegedacht and Roodebloem will again be taken as an example. Let it be assumed that there exists in each case a rainfall record for the period 1911 to 1920, and let it be required to determine the probable annual rainfall for each year from 1910 backwards.

In the first place, the probable true mean must be calculated from the mean for the ten years for which rainfall data are available.

The results have been already given above, viz., Toegedacht 9·80 and Roodebloem 11·06 inches. The estimated annual rainfall required is obtained by adding or subtracting the mean percentage difference from the actual mean for each year of three stations from which the figures for each of the two stations are to be calculated to, or from the calculated means of the stations under consideration. Thus in the case of Toegedacht, taking the year 1901, Table VI shows that the percentage variations from the mean were as follows: Kendrew 21·2+, Cranemere 22·8+, Jansenville 0·4−. The average percentage variation for the area covered by these three stations was 14·5+. The estimated total for the year will thus be 9·80+14·5 per cent; or 9·80 + 1·42 = 11·22 inches. The actual fall for that particular year was 11·35 inches, so that the percentage of error in this particular case amounts to only a little over one per cent.

In Table VIII are shown the results calculated by this method for these two stations, but only for those years for which actual data are available, so that the accuracy of the method can be tested. Very much will depend upon the selection of the stations for which data exists, on which the calculated annual rainfalls are based, and a fair knowledge of the topographical and climatic conditions of the catchment area under investigation is essential. Even with this knowledge a brief study of Table VI will show that the selection is by no means a simple matter, and modifications have to be made for particular years. Thus the deficiency

TABLE VIII

Illustrating Method of Extension of Records of Annual Rainfall.

Year	TOEGEDACHT			
	Actual Recorded Fall	Percentage Deviation from Mean Annual	Calculated Annual Rainfall	Percentage Error
1890	—	—	—	—
1891	—	—	—	—
1892	—	—	—	—
1893	—	—	—	—
1894	—	—	—	—
1895	—	—	—	—
1896	—	—	—	—
1897	—	—	—	—
1898	—	—	—	—
1899	—	—	—	—
1900	—	—	—	—
1901	11·35	14·5+	11·22	1·1 —
1902	11·12	9·3+	10·71	3·69—
1903	6·82	39·4—	5·94	12·9 —
1904	7·38	16·0—	8·23	11·52+
1905	13·37	19·3+	13·64*	2·02+
1906	14·66	40·5+	13·77	6·07—
1907	11·42	4·9+	10·28	9·98—
1908	3·64	38·3—	6·15	69·0 +
1909	11·09	5·8—	9·23	16·77—
1910	7·84	12·6—	8·57	9·31+
1911	11·95			
1912	7·16	Mean for 10 years 8·96 inches		
1913	12·70			
1914	9·58			
1915	3·32	Calculated true mean 9·80 inches		
1916	9·22			
1917	13·98			
1918	5·98			
1919	3·93			
1920	11·80			

* Percentage excess taken equal to mean of Kendrew and Jansenville only. Cranemere omitted.

TABLE VIII (*continued*)

Year	ROODEBLOEM (Based on Four Stations)			
	Actual Recorded Fall	Percentage Deviation from Mean Annual	Calculated Annual Rainfall	Percentage Error
1890	—	—	—	—
1891	18·13	51·5+	16·76	7·6−
1892	14·73	8·8+	12·03	18·33−
1893	12·77	22·8+	13·58	6·34+
1894	11·43	11·9+	12·38	8·31+
1895	15·06	6·1+	11·73	22·11−
1896	12·87	6·4+	11·77	8·55−
1897	9·95	18·6−	9·00	9·55−
1898	12·42	4·8−	10·53	15·22−
1899	7·50	26·5−	8·13	8·4 +
1900	14·09	30·4+	14·34	1·77+
1901	13·94	4·0+	11·50	17·50−
1902	10·38	4·9+	11·60	11·75+
1903	5·32	50·0−	5·53	3·95+
1904	9·63	12·8−	9·64	0·0 +,−
1905	10·96	7·3+	10·25	6·48−
1906	16·47	29·8+	14·36	12·81−
1907	10·31	10·8−	9·87	5·24−
1908	6·15	46·1−	5·96	3·09−
1909	11·83	11·8+	12·37	4·56+
1910	11·60	11·6−	9·78	15·69−
1911	12·72			
1912	5·01	Mean for 10 years 10·01 inches		
1913	14·88			
1914	9·88			
1915	4·79	Calculated true mean 11·06 inches		
1916	9·56			
1917	14·90			
1918	10·07			
1919	3·95			
1920	14·32			

TABLE VIII (*continued*)

Year	ROODEBLOEM (Based on Two Stations)			
	Actual Recorded Fall	Percentage Deviation from Mean Annual	Calculated Annual Rainfall	Percentage Error
1890	—	—	—	—
1891	18·13	50·0+	16·44	9·3 −
1892	14·73	7·5+	11·78	20·03−
1893	12·77	10·5+	12·11	5·17−
1894	11·43	17·5+	12·88	12·68+
1895	15·06	4·5+	11·45	23·97−
1896	12·87	9·7+	12·02	6·6 −
1897	9·95	16·0−	9·21	7·44−
1898	12·42	3·8+	11·38	8·37−
1899	7·50	30·3−	7·64	1·87+
1900	14·09	22·2+	13·39	4·97−
1901	13·94	15·0+	12·60	9·61−
1902	10·38	1·6+	11·14	7·32+
1903	5·32	45·2−	6·01	12·97+
1904	9·63	13·9−	9·44	1·97−
1905	10·96	12·3−	9·61	12·32−
1906	16·47	27·6+	13·98	15·12−
1907	10·31	10·0−	9·68	6·11−
1908	6·15	48·0−	5·70	7·32−
1909	11·83	22·9+	13·46	13·69+
1910	11·60	4·9−	10·42	10·17−
1911	12·72			
1912	5·01	Mean for 10 years 10·01 inches		
1913	14·88			
1914	9·88			
1915	4·79	Calculated true mean 10·96 inches		
1916	9·56			
1917	14·90			
1918	10·07			
1919	3·95			
1920	14·32			

of 20·5 per cent at Cranemere in 1905 is abnormal, and in that year this station was left out of the calculations when determining the rainfall at Toegedacht.

In the northern part of the area conditions are complicated by the very rough character of the topography, and Table VIII shows the annual rainfall for Roodebloem, calculated firstly on the basis of the four stations, New Bethesda, Wellwood, Graaff Reinet, and Winterhoek, and secondly on the two comparatively near stations Wellwood and Graaff Reinet. The results show that the four stations give a closer approximation to the true values than the two neighbouring stations. The results obtained by this method are generally on the safe side. The calculated rainfall in excessively wet years will invariably be underestimated.

The third problem frequently requiring solution is the calculation of missing monthly total falls at a particular station. In this instance any attempt to extend a record backwards over a long period, as was done in the case of yearly totals, is not practicable, but in the case of comparatively short gaps an estimate of this kind is possible and admissible, especially if the calculated daily falls are to be averaged with those of other places.

The method recommended is as follows: Work out for as long a period as possible the percentage differences between the actually recorded monthly rainfall totals of the station with the incomplete record and the nearest adjoining station having a full record and having similar climatic and topographical conditions.

These percentage differences should then be plotted as ordinates with the corresponding monthly rainfall at the reference station as abscissæ. The resulting points will be scattered about in an apparently confused manner, but it is generally possible to draw an average curve through them representing the relation between the two stations.

Such a curve is shown representing the relation between the monthly falls at Jansenville, the reference station, and the corresponding falls at Middlewater, a station approximately forty miles to the south-east. This curve is based upon about eighty comparative readings. For monthly falls of less than 0·7 inch at Jansenville the corresponding monthly falls at Middlewater are in excess. In the case of monthly falls over 0·7 inch at Jansenville the corresponding falls at Middlewater

are smaller. Thus if the monthly total at Jansenville is 2·5 inches, the corresponding monthly total at Middlewater will be thirty per cent smaller. In catchments such as the one dealt with it is often possible to estimate, from the calculated monthly total, the amount which fell during a particular run-off producing storm, which is the result especially aimed at. Such storms are, as will be shown later, not very frequent, and are separated generally by dry periods of considerable duration.

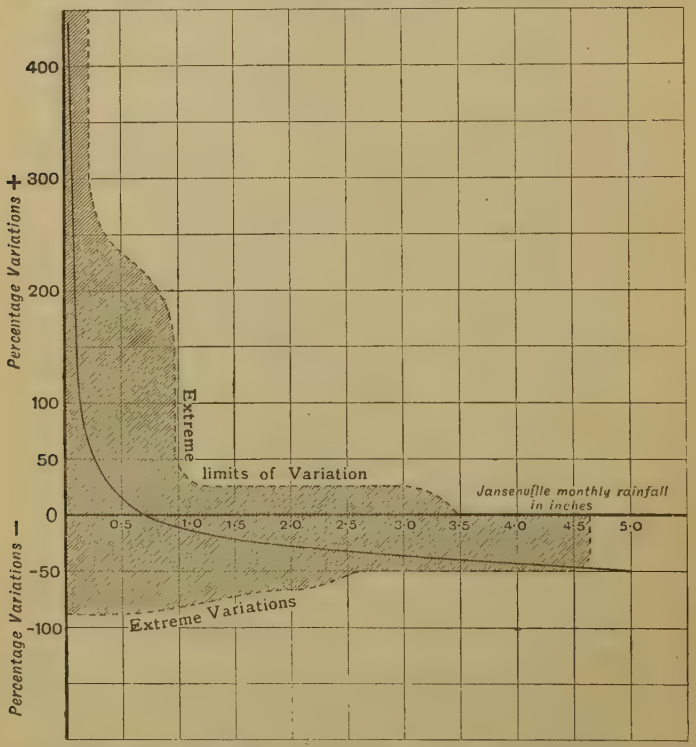


FIG. 5.—Relation of monthly total rainfall of two stations in terms of percentage variation.

In all the above-mentioned calculations the object in view must be borne in mind, viz., the study of the rainfall conditions over a catchment area in relation to the run-off derived therefrom or the stream flow due directly or indirectly to that rainfall, and when no better data than the annual rainfall are available, errors of ten or fifteen per cent in the case of annual totals

of ten inches upwards are of no great importance. In the case of very small annual totals the percentages of error give an exaggerated idea of the variation between the calculated and actual fall.

There is another method by which a short record could be extended, provided that there are a sufficient number of other stations with long records in the vicinity. The method consists of drawing isohyets, or lines of equal annual rainfall, on a map of the country for each year or month, and estimating the desired rainfall for the place in question from its position on the map in relation to the isohyets. This method is not likely to yield any more accurate results unless a large quantity of data are available, which is rarely the case.

For the calculation of the run-off, in the absence of more detailed data the mean annual rainfall over a catchment or portion thereof is required, and in calculating these catchment means it is most desirable that the same stations be used for each year. If one of the records cease it should be extended in the manner described above, so long as there are a sufficient number of other stations to justify the assumption that they are still more or less representative of the whole catchment. The question now arises: How many rain gauges are necessary in a catchment area of a given size to furnish, approximately, a true mean of the rainfall over that catchment ?

It will be shown in a subsequent chapter that to arrive at the true relationship between rainfall and run-off continuous rainfall records at innumerable stations all over the catchment would be necessary. Such data are never available, and the engineer must satisfy himself as to what minimum rainfall data in a catchment are necessary to enable him to calculate, approximately, the annual run-off.

For the purpose of making accurate and continuous gaugings of the run-off from certain selected catchments it is customary for the Union Irrigation Department to supplement the rain gauge stations of the Meteorological Department by a large number of special gauges carefully distributed over the catchment area. Thus, for example, on the Riet River, in the Orange Free State, with a catchment area of 4,542 square miles, sixty-nine special gauges were established ; on the Stormberg River, with a 970 square mile catchment, forty-two gauges ; and on the Tarka River, with a 1,885 square mile catchment, forty gauges. Now if we only wish to concern ourselves with

the estimated annual run-off from a catchment for which no stream flow gaugings exist, such a multiplicity of rain gauges is unnecessary and a lesser number will suffice, provided they are selected with great care and are thoroughly representative of the normal rainfall conditions in different portions of the catchment.

This can be illustrated by a typical example. The Tarka catchment in the Cape midlands is gauged by the Hydrographical Branch of the Irrigation Department, and there are forty-two rain gauges distributed over 1,885 square miles. There are, however, only three gauges of long standing reporting to the Meteorological Department which can be considered as representative of the catchment, viz., Hofmeyr, Tarkastad, and Cradock, and the last named is some distance outside the boundary of the catchment. Now, in the years for which it is possible to make a comparison we find that the annual catchment averages represented by the forty-two and the three gauges respectively are as follows : 1914 to 1915, 12·81 inches and 12·78 inches ; 1915 to 1916, 12·90 inches and 13·15 inches ; 1916 to 1917, 18·59 inches and 20·42 inches ; 1917 to 1918, 1934 inches and 20·38 inches. It is clear from the above figures that for all practical purposes the three carefully selected gauges are sufficient to furnish a representative value of the average catchment rainfall for each year. It must be borne in mind, however, that this would not be the case had the three gauges been injudiciously chosen.

It often happens, especially in large areas, that rain gauges are very unequally distributed. In all catchments which contain a considerable percentage of mountainous country there are large and very important portions which are undeveloped, and therefore settled to a very limited extent or perhaps not at all. As has been already stated, rain gauges in South Africa are located only in the more developed parts of the country, and it may therefore happen that highly important sections of a catchment are entirely devoid of gauges. This often leads to very erroneous estimates of the mean rainfall over a catchment. Topography may or may not exercise a very marked influence on the precipitation. The relation between rainfall and topography depends upon the conditions under which rain, in general, is induced. When the rain-bearing winds from the ocean strike mountain ranges, as is frequently the case with the great coastal ranges and escarpments of South Africa,

the rainfall gradient is often very steep, and the annual precipitation may increase fifty or even a hundred per cent with a rise of 1,000 feet.

On the plateau and within the thunderstorm area the effect of topography on the rainfall is very indefinite and often is non-existent. This is due to the fact that ordinarily thunderstorm rain originates in South Africa at enormously high altitudes. Nevertheless many instances of such influence are apparent in various parts of the country. Thus in a recent analysis by the Chief Meteorologist for the Union of the rainfall over the Vaal River catchment, which covers an area of 75,000 square miles, it was found that there existed a number of detached areas of high rainfall bearing an evident relation to the topographical conditions—" a relationship totally unexpected." The assumption that, with the area of thunderstorm rainfall, topography has no influence whatever is not a safe one, and when investigating the rainfall of an important catchment steps should be taken, wherever possible, to locate rain gauges in the higher mountainous parts of the drainage area.

On the other hand, in the absence of strong evidence of such influences, no theoretical assumptions of increased rainfall with altitude should on any account be made in the regions in question.

The mean of all the rain gauge stations within a large catchment will rarely be representative of the whole area. Specially favoured small areas which happen to be closely settled will be over-represented, whereas the less favoured portions will be greatly under-represented. In such cases it is either necessary to eliminate a number of stations in the over-represented portions and be satisfied with a fewer number of well-selected stations, or to divide the catchment into sections and calculate the average for each station separately.

There are numerous other important matters concerning rainfall which can be more conveniently referred to in the subsequent chapters dealing with run-off, maximum floods, and conservation.

CHAPTER III

FLOW OF RAINFALL FROM THE GROUND OR RUN-OFF

WHILE moisture precipitated from the atmosphere forms the source of practically all water supplies used for irrigation, town supplies, or water power schemes, only a fraction of the amount which actually falls on the ground as rain, snow, or hail is available for those purposes. The most important question which the water engineer has to deal with is to determine, or, at least, estimate, how much of the total precipitation can be collected and made use of.

Of the rain or snow falling on the ground a very large percentage is evaporated again at the surface. It is not only water lying on the surface which is evaporated, but also water which has penetrated into the porous soil and is subsequently brought to the surface again by capillary action. Much water is likewise absorbed by vegetation, and the volume taken up by various plants and transpired and evaporated is enormous. In dry climates it may reach as much as 900 lbs. of water for each pound of dry plant matter produced.

Experiments have shown that during the growing season cereal crops absorb from fifteen to twenty-six inches of water. Timber will absorb nine to twelve inches and grasses may absorb about twenty-six inches.

The average evaporation from a moist bare earth surface is about two-thirds of that from a water surface in a tank, whereas from a moist surface covered with short grass it is about ninety per cent greater than from such a water surface.

A considerable volume of water soaks into the ground, and under the influence of gravitation sinks into the subsoil. Some of this subsoil water is again given off by springs, and it is this underground supply which enables stream flow to be maintained during protracted periods of drought and light rainfall.

Water which cannot emerge from springs can only be reached by means of wells and boreholes.

The volume of water which is discharged from any catchment or drainage area varies enormously. Such variations depend partly upon the physical peculiarities of the drainage area, partly upon general climatic conditions, and partly upon the manner in which the moisture is, in the first instance, precipitated from the atmosphere.

For any given catchment the first two causes of variation are more or less constant, though even in this case the same catchment is subject to certain important variations.

The third cause of variations, viz., mode of precipitation, is a most unstable factor, and is the main cause of the large fluctuation in the run-off from a given catchment.

The main physical characters influencing run-off are geological structure, permeability of the soil, and steepness of the slopes.

The chief climatic influences are character of the vegetation which covers the surface, comparative dryness of the climate, and especially the length of the drought intervals between run-off producing rains.

The conditions of run-off from two or more catchments are only comparable if the physical and climatic conditions are similar. These conditions vary to such a great degree, however, and there are so many unknown or uncertain factors which come into play, that really typical cases are very difficult to find.

For water engineers the problem is to deduce the run-off from a certain catchment area from the rain which falls over that area, and which is known with varying degrees of accuracy. For the present we are only concerned with the volume of water which flows off during a considerable period of time, for example, per month or per year.

Engineers have employed various methods to express the relationship between the run-off and the rainfall. The most common is the PROPORTIONAL METHOD, which consists in calculating the proportion of run-off to rainfall for each year, month, or other period for which observations were taken, and obtaining the run-off in other years by multiplying the rainfall for each month by the corresponding proportion obtained.

Another method which has received considerable attention is due to Vermeule, and this takes into account the effect of ground storage water on the run-off. This method is more

rational than any of the others, but is complicated, and it depends upon the accuracy of certain broad assumptions. For a proper understanding of the subject it is most advisable to study carefully Vermeule's method.

In view of the great importance of vegetation in its effect upon evaporation the water year or hydrographic year may be conveniently divided into two periods: firstly, the growing period, which, over the greater part of South Africa, is during the summer and during the wet seasons; and secondly, the remainder of the year, during which evaporation is comparatively small.

Thus Vermeule, as the result of many investigations in the Eastern United States has derived the following expression:

E=Evaporation in inches=15·5+0·16×Rainfall in inches for the whole year.

E=4·2+0·12 R. for the winter period.

E=11·3+0·20 R. for the growing period.

These formulæ are given as examples only, and would be quite inapplicable to South African conditions.

If such formulæ could be worked out for, say, Transvaal High Veld conditions, they would not be applicable to the western province where the rainfall seasons are reversed.

For the purpose of taking ground storage into consideration, Vermeule divides catchments into three classes, viz.:

1. Highland areas of bold relief and overlying impermeable strata.
2. Areas overlying permeable strata with no lakes or swamps.
3. Areas overlying permeable strata with large swamps or surface storage.

For each of these classes he has constructed curves the ordinates of which represent the monthly flow in inches from the ground storage, while the abscissæ represent the corresponding depletion of the ground storage, likewise in inches.

It may be stated at once that, for the present at any rate, Vermeule's method cannot be applied on anything but a very small scale in South Africa. Conditions in this country are far too erratic, and we possess far too few data for the purpose.

Whatever method is adopted, the results obtained are likely to be rough approximations only. The actual run-off depends upon such a multitude of conditions, many of which are variable, that the problem is not capable, in practice, of mathematical

treatment. So far as practical engineering is concerned there are no such things as laws of run-off.

A great deal of elaborate research work in this connexion has been carried out in temperate and humid countries, such as England, Germany, France, Switzerland, and the Eastern United States, and it is very necessary to remember this fact when reading up this important subject in European and American technical books and papers. Compared with the difficulties with which the engineer has to contend when studying the problem in a hot and semi-arid country the parallel problem in England or Germany is simple. Furthermore, in the old and highly developed countries of Europe precise data are available over a much longer period, and the physical condition of the drainage areas in those countries is more or less fixed.

In countries like South Africa or Australia data are scanty and often unsatisfactory, and from various artificial causes the character of the surface of very large areas has changed materially within historic times. So far as South Africa is concerned the engineer must avoid, so far as possible, all empirical formulæ or arbitrary percentages which may be found in technical books published elsewhere. The use of such methods will surely lead to trouble.

A method of calculating the run-off from a catchment, which is both useless and mischievous, was until recently popular amongst engineers in South Africa. This method consisted in calculating the mean annual rainfall over the whole catchment from such data as existed. These mean rainfalls were often derived from records of very varying lengths and in respect of badly distributed stations. Having obtained what was considered the mean rainfall for the area, an arbitrary percentage representing the run-off was then applied, and the result was accepted as the available annual volume of discharge.

A more dangerous and misleading method of calculating run-off can hardly be imagined. It is an excellent method of ensuring the failure of a large irrigation or waterworks project.

Before going further with this subject it is necessary to deal with the subject of units employed in calculations of this kind.

We are here dealing with very large volumes, and it is most desirable to adopt units which are not unnecessarily cumbersome. There is a very large possibility of error when making extensive calculations in thousands of millions. Decimal points are easily misplaced and noughts easily omitted. The smaller the working

figures the more convenient and accurate are likely to be the calculations.

In countries using metrical measures the standard units of water measurement are litres per second, or cubic metres per second, as units of discharge, and cubic metres as the unit of volume. Rainfall is measured in millimetres.

In countries using English measures two fundamental units are employed. The waterworks engineer uses the gallon. This is mainly due to the fact that the basis of his calculation is the quantity of water allowed to each individual in a town or area of supply, and this quantity being relatively small the gallon unit is convenient. When it comes to large storage problems the gallon is most inconveniently small, and it becomes necessary to talk of billions of gallons and other magnificent figures.

When the waterworks engineer deals with discharges he adopts minutes, hours, and days as units of time. As a result of this pump capacities are generally given in gallons per minute. Now that English-speaking engineers use English and American technical textbooks, etc., with equal confidence, the difference between the English "imperial" gallon and the United States gallon, which is only 0·8 of the imperial gallon, must be ever kept in view. Confusion between English and United States gallons has caused many serious mistakes.

For the irrigation engineer the gallon is much too small a unit; thus one light watering of an acre of land consumes approximately 70,000 gallons.

Irrigation and water-power engineers have adopted the cubic foot as the fundamental unit of volume, and the cubic foot per second as the unit of discharge. This latter measurement has been abbreviated to "CUSECS" in India, and this term is now used largely in South Africa and elsewhere. In the United States the corresponding abbreviation is the second foot, or "sec-ft."

When dealing with large volumes this unit is still inconveniently small, and a larger one has come into use, viz., the acre foot, which represents 43,560 cubic feet. This sounds somewhat awkward but is not really so.

The acre is the most common unit of area, and forms the basis of nearly all irrigation and agricultural statistics in the British Empire and in the United States. An acre foot, therefore, provides an immediately comprehensible relationship between the water supply and the land to be watered. There

is, however, another important, even if quite accidental, link between the two.

There are in the twenty-four hour day 86,400 seconds. This is very nearly equal to the number of square feet in two acres, viz., 87,120. The difference amounts to less than one per cent. It is thus apparent that one cusec flowing continuously for twenty-four hours will cover two acres almost exactly one foot deep ; or, if we are thinking in terms of, say, three inch irrigations, one cusec flowing continuously for twenty-four hours will (excluding transit losses) of course irrigate eight acres. For rapid approximate calculations this relationship is useful.

When dealing with run-off and storage one acre foot, or, alternately, one million cubic feet, are the most suitable units. One million cubic feet is equal to 22·957, or, say, twenty-three acre feet.

Run-off is expressed either as a monthly or annual volume from the entire catchment, or from one square mile thereof, or as cusecs per square mile continuous discharge over a given period.

Reverting now to run-off in South Africa, we are mainly concerned with the following types of catchments :—

Firstly.—The arid areas of the North-Western Cape, Gordonia, etc. The country is largely granitic, is generally flat, and is to a great extent covered by coarse sandy soil.

Secondly.—Mountainous or hilly areas, largely impervious, with little soil covering and little vegetation of the Karroo type. Such areas are typical of a very large proportion of the Karroo in the Cape and Orange Free State Provinces. The geological strata are largely composed of shales and sandstones, intersected or partially capped in places with dolerite dykes or sheets. The surface soil in these areas is thin and frequently underlain by layers of calcareous tufa. Alluvial soils are clayey, and after a heavy shower are apt to puddle on the surface and become impervious. In such catchments the ground storage is very small.

Thirdly.—Similar country to the above from a geological point of view, but, owing to more favourable and more regular rainfall, better covered with vegetation, chiefly grasses, and carrying a greater depth of soil covering, which is lighter and deeper than in the first class. Such areas are to be found in the eastern portions of the Cape and Free State, and eastern Natal.

Fourthly.—The Transvaal high veld, other than that on the dolomite formation. The country is largely flat, sandy and absorbent, and entirely grass covered. Trees are scarce.

Fifthly.—The Transvaal middle or bush veld, which is largely granitic. It is thickly covered with grass, bushes, and trees. The soil is deep and sandy.

Sixthly.—The dolomite areas of the Transvaal and Bechuanaland. This country is generally flat and is well covered with grass. Soil covering is sandy but not deep. The rock is open jointed and full of fissures, swallow holes, etc.

Seventhly, and finally.—Mountainous areas, more especially the coastal zone from Cape Town, round the south and east coasts, between the great escarpment and the sea. This is generally a region of heavy rainfall ; the escarpment mountains, which attain very high altitudes, are steep and very broken, and the zone is generally covered with very dense vegetation.

These divisions are drawn on very broad lines and are not intended to be exhaustive. To attempt such a classification would be quite beyond the scope of this chapter.

It is not intended to deal here with all these areas, as some of them are comparatively unimportant to the irrigation engineer.

Thus the arid area comprising the first class produces practically no run-off at all. Heavy storms on the harder surfaces produce run-off, but the effect is local and the flow soon becomes absorbed. Speaking generally, no rainfall over any part of this area laying more than a hundred miles from the coast ever reaches the sea.

Mountainous areas, comprising Class No. 7, will also not be dealt with. The conditions of run-off from these areas are highly complex and variable, and have been studied in respect of very small areas only. These catchments, to a large extent, lie outside the area in which irrigation engineering problems involving the study of run-off have to be dealt with, but are of importance when studying water-power schemes.

The areas which are of chief importance, and which will have to be dealt with, are those comprised in Classes 2, 3, 4, and 6.

Class 2 is the most important of all at present. The area includes all the best portions of the Cape and Free State Karroo, and has absorbed, and for a long time to come will absorb, most of the energies of the irrigation engineers. Furthermore, in no other area is the problem of run-off more complex.

It is only in very recent times that the problem of the rela-

FIG. 6.—Hydrographic Survey Gauging Weir on Crocodile River, Central Tràns vaal.

FIG. 6A.—Hydrographic Survey Gauging Weir on Nels River, Cape Province.

FIG. 6B.—Hydrographic Survey Gauging Weir on Nels River, Cape Province.

tionship of run-off to rainfall has been systematically and scientifically investigated in South Africa. The proper gauging of stream flow is a very costly undertaking and requires expert handling.

Irrigation Departments were established in the Cape in 1904, and in the Transvaal a little earlier. Prior to union the Cape Irrigation Department never had sufficient funds to undertake this important branch of work. The Transvaal was more fortunate, and a properly organized hydrographic branch was started about 1905, liberally supplied with funds. After union the opportunity came to extend operations outside the Transvaal, and progress continued till about 1916, when the financial stringency of war-time was responsible for the cessation of further development.

The position to-day is as follows :—

In the Transvaal seventeen gauging stations have been established, at which the discharges of rivers are gauged continuously by means of clockwork apparatus, the flow being measured over substantially constructed permanent gauging wires. From each station a continuous chart is obtained of the flow in the river. Each catchment is provided with a large number of special rain gauges, carefully distributed over the area. The records of these gauging stations extend over varying periods, the oldest being started in 1904. The most recent gauging station in the Transvaal was established in 1909.

Besides these seventeen gauges there are several others which were established by the Rand Water Board under Government supervision in 1915, and one Government station was abandoned after it had served its purpose. In the Cape Province eight first order stations and several second order stations were established, four of which have been abandoned. For the Orange Free State there are three first order and several second order stations.

Apart from the results obtained by the Hydrographic Survey Department of the Government there are very few data available which are of any value, and the official record, short as it is, furnishes practically the entire basis for the study of the run-off problem in South Africa.

From a particular catchment the run-off will vary enormously according to the manner in which rain falls thereon. The total rainfall for the year gives no indication whatsoever of the manner in which the rainfall was received. If the total run-off

is known, this volume, expressed as a percentage of the annual rainfall, gives some very erratic results. Thus practically the same annual fall of approximately 22 inches over 1,500 square miles of the Great Olifants River catchment yielded percentages of 1·31, 0·41, and 3·91 respectively in different years.

On the Riet River, in the Free State, 16·73 inches of mean rainfall over the catchment yielded 3·65 per cent in one year, and practically the same mean rainfall during the following year yielded only 0·98 per cent. Such examples can be multiplied indefinitely.

In the dry areas, where thunderstorm conditions prevail, one reason for unexpected results is the occurrence of heavy and very local storms, which, if near the gauging station, may cause a big flow to be recorded and yet escape registration at any of the numerous rain gauges. On most of the observed catchments this cannot be of frequent occurrence, as the rain gauges are sufficiently close together to prevent omissions on a big enough scale to affect the percentage for the whole year, though they materially affect the result if calculated for an individual storm.

The chief causes of the inconsistency in the percentages are the erratic distribution of rainfall over any large area and the great variation in the intensity and precipitation. Half an inch falling in a few minutes may produce a flood, whereas an inch falling as a drizzle, and lasting all day, may produce no run-off at all.

The condition of the catchment during a period of rain is also of great importance. The surface may be parched and baked after a long drought during the hot season, or it may be wet or saturated from previous rains. The same fall of rain on the same catchment in these two different states will produce vastly different results. Fluctuations of this particular kind are more marked in arid regions than in more humid areas.

Storms of similar intensity will produce different results during the winter months, when evaporation is low and vegetation is not growing, than in the summer, when vegetation is growing luxuriantly and surface evaporation is high.

Generally speaking, the following combination of conditions will produce a very high run-off, and, under extreme conditions, a great flood :—

A rainstorm of great intensity and of more or less equal force all over the catchment, the rain falling on an already

damp or saturated surface. If the drainage area were long and narrow, a storm travelling down the long axis of the catchment would produce a greater flood than one operating all over the catchment at the same time.

All the greatest floods, particulars of which are known, were produced under conditions such as these.

To give an example : early in May, 1916, a destructive flood of great size passed down the Sundays River. This was due to a severe and widely distributed storm which lasted two days. On the first day the entire catchment received falls varying from 0·6 inches to 2·36 inches, the mean fall over the catchment being 1·32 inches. On the second day the fall was heavier, the individual falls yarying from 0·82 to 3·42 inches, and the mean for the whole area of 6,000 square miles being 2·11 inches. The flood resulting from this storm amounted to 5,500 million cubic feet, or about twelve per cent. of the total fall over the catchment. Prior to this storm there had been no rain over the catchment for twenty-seven days.

In December, 1921, a flood of even greater intensity resulted from general rain over the entire catchment, lasting six days and falling on an already damp surface. The flood resulting from this storm yielded 13,000 million cubic feet, or about twenty-six per cent of the total rain falling over the drainage area.

On the other hand, the following conditions will give rise to a very poor run-off :—

Scattered and intermittent rains of small intensity and long spells of dry weather between the storms.

When a catchment is in a dry state it will absorb a considerable fall of rain before any of it reaches the main drainage lines. The minimum run-off producing fall varies on the same catchment according to the season of the year and the degree to which the surface has been dried out.

In ordinary practical problems it is only possible to base calculations on the mean fall over the whole catchment, and when conflicting results are obtained in this connexion it is generally due to the run-off being influenced by the fact that the mean fall over the whole catchment is not truly representative of the summation of all the falls at innumerable spots all over the area.

Experience has shown that on dry catchments mean falls of less than 0·5 of an inch rarely produce run-off, and in many cases this minimum figure is very much higher, especially if

TABLE IX

Analysis of Daily Rainfall at Certain Places in the Drier Parts of the Cape Province. Classification of Falls is shown as Percentages of the Aggregate Fall and of Total Number of Rain Days throughout the Period Analysed.

Name of Station	Length of Record Analysed	Mean Annual Rainfall	Aggregate Rainfall in Inches	Aggregate Number of Rain Days	0 to 0·49		0·50 to 0·99		1·00 to 1·99		2·00 and over	
					Inches	Days	Inches	Days	Inches	Days	Inches	Days
Kamfers Kraal ..	32	8·27	281·84	1,157	50·3	85·5	29·2	—	18·2	—	2·3	—
Brakfontein ..	38	10·02	366·28	1,301	47·0	83·0	29·9	—	20·8	—	2·4	—
Jansenville ..	43	10·74	461·78	1,737	47·42	80·5	28·5	11·2	23·7	4·7	0·43	0·06
Aberdeen ..	42	12·28	511·74	1,690	41·03	80·9	29·7	13·5	19·5	4·4	9·7	1·20
Graaff Reinet ..	42	13·88	577·39	2,241	45·11	84·74	28·6	10·9	20·0	3·7	6·3	0·62
New Bethesda ..	37	14·00	503·22	2,340	38·83	85·85	29·91	9·57	23·01	3·85	8·25	0·73
Winterhoek ..	32	17·14	527·67	1,571	40·20	79·89	27·14	13·43	22·27	5·41	10·39	1·27
Jansenville (1915)*	—	—	4·48	26	49·5	81·0	21·7	11·5	28·8	7·5	—	—
Do. (1919)*	—	—	5·07	19	34·5	63·2	26·0	21·0	39·5	15·8	—	—
Do. (1908)*	—	—	5·35	26	50·5	80·8	16·8	11·5	10·3	3·8	22·4	3·8
Do. (1893)†	—	—	17·90	71	20·3	70·4	19·5	12·7	36·2	12·7	24·5	4·2
Do. (1891)†	—	—	17·36	68	22·2	61·8	35·8	25·0	23·7	8·8	18·3	4·4
Do. (1917)†	—	—	17·31	39	11·7	48·7	13·3	15·4	39·0	25·6	35·9	10·3

* Three driest years on record. † Three wettest years on record.

water is being diverted into irrigation furrows, which introduces a very disturbing factor. Broadly speaking, it is general falls in excess of half an inch which produce stream flow.

Let us see, now, in what manner the rain is received during individual storms, in so far as quantity is concerned.

For this purpose the rainfall of seven stations in the Karroo and Cape midlands has been analysed over long periods of time, with the results shown in Table IX.

At the following five places, viz., Kamfer's Kraal, Brakfontein (both in the Beaufort West district), Jansenville, Aberdeen, and Graaff Reinet, no less than forty-six per cent of the total rainfall was received in showers of less than half an inch, and such falls accounted for approximately eighty-three per cent of the total number of rain days. A further twenty-nine per cent of the total rainfall was received in showers of from half to one inch.

It will thus be seen that over a very great area of the country in question half of the rain which is received annually is generally incapable of producing any run-off. When the mean of a number of stations is taken the position is even worse. In the dry regions it is the falls over one inch which produce useful run-off, and in the case of the stations examined these amount to only twenty-five per cent of the total fall. If this analysis is carried farther, and the conditions obtaining in wet and dry years examined, the position is shown to be more unfavourable in dry than in wet years. Thus, taking the three driest years on record at Jansenville, the percentages of the total rain received, falling in showers of less than half an inch, were seventy, sixty, and sixty-eight respectively, whereas in the case of the three wettest years, the corresponding percentages were only forty, fifty-eight, and twenty-five respectively.

Rainfall is measured daily in all official rain gauges in South Africa at 8.30 each morning. This is a time of the day when there is usually a break in the rain, but many individually recorded falls are really continuous over two rain days; that is to say, rain may commence at 6 a.m. and continue without material interruption until 10 a.m. The total fall might be 0·75 inch, but it would appear in the daily rainfall record as two falls on successive days of, say, 0·40 and 0·35 respectively. As individual falls they might be rejected as run-off producers, but considered as one fall, such a storm would probably produce considerable run-off.

It is thus necessary to use some judgment when working with the daily rainfall records and estimate in every case what constitutes an individual fall, whether this be spread over one day or two or more days. With a little experience this can be done within reasonable limits of accuracy.

It occasionally happens, even in the drier parts of the country, that daily light rains are received over a considerable period of time and over a large area. More or less continuous rain of this kind, if accompanied by constant cloudy weather and an absence of wind, may give rise to a high percentage of run-off, though the individual falls are small, or at best only moderate. A good example of this took place during February and March of 1920 in the Sundays River catchment.

During this period rain fell at most of the stations in the catchment, on twenty-seven days, distributed as follows: From 29th January to 10th February, on the 13th and 14th February, on the 20th and 21st and the 26th and 27th February, and on the 12th to the 16th and 20th to 22nd March. Most of the rain was very light, falls of over one inch being very few and at isolated stations. The result of this long period of rainy weather was to produce 8,200 millions cubic feet of run-off equal to 7·3 per cent of the rainfall, with two sharp floods of twelve hours' duration, each early in February. In this case the catchment became thoroughly saturated, and the effect of ground storage became considerable. Such rainfall conditions in the Cape midlands are, however, rare.

While the total amount of rain recorded as falling in one day is, on the whole, a fairly good guide as to the run-off producing capacity of the day's rain, the really important factor in the matter is the intensity with which the rain falls.

Unfortunately, information regarding the intensity of any particular fall is, with very few exceptions, entirely lacking. Observers sometimes make notes which throw a little light on the subject, and this is nearly always done in the case of very exceptional storms which happen to impress the observer. What we really require to know is the rate at which the rain falls. Such information can only be furnished by a self-recording rain gauge. These instruments are delicate and costly, and only very few of them are supplied by the Hydrographic Survey Department to competent and trusted observers.

When the daily charts of such an automatic gauge are examined the great importance of this intensity factor becomes

obvious. An inch of rain may be precipitated in a few minutes, when it would be popularly described as a "cloud-burst," and yield a very high percentage of run-off; or it may take, say, two or three hours to fall, which might still be described as heavy rain and yield a good percentage of run-off; or it might take the whole twenty-four hours, and yield practically no run-off.

In order to establish any true relationship between run-off and rainfall it would be necessary, in the first place, to provide an enormous number of automatic recording gauges, which in practice is impossible. Sufficient competent observers are not available, the labour of computing would be enormous, and the cost very great. In any catchment area of appreciable size in the more arid regions the installation of one or two automatic gauges would help but little, as there are very large local variations in the intensity. Exceptionally heavy downpours are generally very local indeed. The record of one automatic gauge, therefore, can not be accepted as representative of anything but a very small area in the neighbourhood of the apparatus.

Under the circumstances we must be content with something far short of the ideal. The greater part of the rain in the areas with which we are now concerned falls during thunderstorms and as heavy showers, often followed by a spell of steady "soft" rain. Here and there a deluge occurs over a small area but the average over a large area is not greatly affected, and it is with average conditions that we must be content. Working on the basis of such average conditions will tend to underestimate the run-off rather than over-estimate it. It is, more particularly, when we desire information regarding the maximum run-off from a catchment that detailed information about maximum intensities becomes a matter of greatest importance.

So far we have dealt only with the difficulties encountered, and it would almost appear as if the problem were so complicated and the data so inadequate, that its solution is impossible. It is obvious, however, that if the engineer is going to throw up the sponge, then all progress with irrigation, water power, and domestic water supply becomes impossible, unless each scheme is preceded by a lengthy period of actual gaugings. This attitude would be unjustifiable and contrary to all national interests.

If we cannot estimate accurately the yield from a catchment we can at least arrive at an approximate estimate, and the engineer's task resolves itself into one of making his estimate in such a manner that it shall be as near the actual truth as

available data, experience, and sound judgment render possible. As a failure to obtain the results predicted is likely to lead to serious financial loss and distress, estimates must not be framed in an irresponsible and unduly optimistic manner. Ultra-caution is, however, equally to be condemned, as it places an unjustifiable check on progress. Some risk must be taken; the problem is to minimize the risk as much as possible.

The essential problem for the engineer is to frame a reasonably accurate forecast of the run-off from a catchment at any particular spot in the future. Such a forecast can only be made by assuming that the rainfall and run-off conditions which have been proved to have existed during a reasonably long period in the past will continue to operate in the future. This is not always the case, as in many parts of a new country, especially a dry one, such as South Africa, the surface conditions have changed appreciably during the past century as the result of settlement. The evils of desiccation are an inevitable accompaniment to the settlement of a new country. Destruction of vegetation and erosion proceed with alarming rapidity, and in the drier parts of South Africa large tracts of country have been ruined by these processes. The hydrographic effects of desiccation are the elimination or curtailment of ground storage, the conversion of perennial streams to intermittent streams, the increase in the rate of run-off, and probably an increase in the total volume, or, at any rate, the percentage of run-off owing to the elimination of the natural vegetation.

In the present chapter we are less concerned with the rate of run-off than with the total volume of water appearing as stream flow, and as our run-off data are all of recent origin and the condition of the surface of the country is not likely to alter materially during the next thirty years we are justified in predicting the total run-off during future years on the basis of the data furnished during the past twenty-years. The stream flows thus predicted will err on the side of caution. We may assume that the evils of desiccation have passed their worst phase. The country is alive to the dangers arising thereform, and the increase in rural population and agricultural development tends to restore, in a better form perhaps, the surface conditions of the past in so far as they affect the run-off.

The development of irrigation has a particularly important effect on the flow of streams, and will in course of time greatly improve the conditions of many of the rivers of South Africa,

and should even enlarge the scope of irrigation and agricultural development beyond what now appears practicable. A few brief remarks on this important subject will not be out of place here.

A universal result of irrigation is the increase in the permanent stream flow below extensive areas of irrigated land by what is known as "return flow water" or "return seepage." Later on in this book, when dealing with the duty of water, the design of distribution systems, drainage water, etc., stress is laid on the evil effects of over-irrigation, but, quite apart from this, it is in practice impossible to deliver only the minimum quantity of water on to the land which will ensure the production of a good crop. A small excess of water discharged on to the land is inevitable. Then, again, it has been shown in the chapter on absorption losses in canals that where these are unlined, which is generally the case, a very large percentage of the water taken into the canal system at the head works never reaches the lands to be irrigated at all, but is lost by absorption or seepage into the ground through the sides and bed of the canals, water-courses, etc.

All this excess water taken by a canal system is, from the point of view of efficiency of the system, waste; but if we consider the question in its larger aspect, a large percentage of this waste is recovered, and seepage losses can only be described as objectionable and harmful if from over-irrigation and want of adequate drainage facilities the efficiency of the irrigation system is impaired from agricultural and commercial points of view.

The fact that a very considerable amount of the water diverted from a river for the irrigation of land in the same drainage area is ultimately returned to the river in the form of seepage and surface drainage is well known, and in South Africa many striking examples of this phenomenon exist, even though the areas irrigated and furnishing this return seepage are small in comparison with similar areas in other countries. In countries where there is a great shortage of water for irrigation and even domestic purposes, as is the case in South Africa, return seepage is a matter of the greatest importance, as without it, under the present conditions of somewhat wasteful irrigation, which are inevitable in the earlier history of irrigation, the carrying out of large schemes in the upper parts of a river system would seriously prejudice the possibility of developing portions of the same catchment lower down.

The beneficial effect of return seepage in improving the permanent flow or even creating such a flow is realized in a vague sort of way, and in South Africa the courts when apportioning water to various schemes along a public stream, now invariably take this important fact into account. It is clear however, from the evidence given in these courts regarding return seepage that little or no definite information is available on the subject. In this connexion countries depending upon irrigation owe a great debt of gratitude to the numerous investigators in the western United States, who have for many years past been studying this important problem.

Much useful information has been published by the United States Department of Agriculture concerning the measurements of return seepage in the South Platte River and its tributaries in Colorado, these measurements extending back to 1885 in the case of the Cache la Poudre River, and to 1889 in the case of the South Platte River.

The available measurements in the western United States have recently been summarized and analysed in an article by R. I. Meeker in the "Engineering News-Record" (Vol. 89, No. 3, July, 1922, page 105), and a summary of the conclusions arrived at are here quoted in full :—

(1) Ordinarily fifty per cent of the water diverted for irrigation purposes becomes a source of return flow.

(2) Annual return flow of from thirty-five to sixty-five per cent of the river flow diverted have been measured for large, compact irrigated areas.

(3) In the South Platte and tributary valleys in Colorado, where 1,100,000 acres are irrigated, return flow amounts to 1,000,000 acre feet annually.

(4) In the Cache la Poudre valley, where irrigation is intensive and 250,000 acres are irrigated, the annual return flow is 130,000 acre feet, or about 0·5 acre feet per acre.

(5) In the North Platte Valley, Nebraska, where water is plentiful, the annual returns from 250,000 acres are 1·6 acre feet per acre.

(6) Annual returns from the two Colorado projects in the Carmel and Rio Grande drainage districts of 5,000 and 30,000 acres respectively are approximately 1 acre foot per acre.

(7) Monthly returns throughout the year are not constant.

The summer and autumn months yield the maximum amount of return seepage. The minimum returns occur during the winter and spring months.

(8) From fifty to sixty per cent of return flow under natural or artificial drainage occurs during the irrigation season and is available for re-use.

(9) Return flow waters from irrigation in the older irrigated valleys are a large factor in the water supply and have a large economic value.

It is obvious that such large amounts of return seepage are only possible if the irrigated areas are large and compact, and that the under drainage is not complicated by any geological peculiarities such as exist, for example, in the dolomitic areas of South Africa.

It may be assumed, however, that under normal conditions obtaining in the Union a return of twenty per cent may be reasonably anticipated in most of the irrigation areas, and that after the first accumulation of alkali salts have been leached out of the soil and carried away, this large flow of water will become available for perennial irrigation ; in other words, the irrigation of one hundred acres will after the lapse of some years make it possible to irrigate a further twenty acres which would otherwise remain uncultivated.

We must now first endeavour to establish a relationship between rainfall and run-off in respect of catchments which have been under investigation, and which can be applied to the same catchments or to others regarding which no data exist, so that estimates of the run-off in future years can be made.

When estimating the run-off from a catchment the extent of the drainage area is a matter of very great importance. Given a catchment of, say, one square mile of symmetrical shape and uniform character, and very full data regarding the rainfall, it should be possible to predict the run-off with fair accuracy.

The bigger the catchment and the more diverse the topographical features the more complicated and uncertain does the problem become. It is unfortunate that most of the catchments which have been dealt with by the Irrigation Department are of large area, the majority of them varying from 1,000 to 6,000 square miles. There are only three catchments with an area of less than 100 square miles, which have been investigated and then were abandoned after short periods of gauging. This has been unavoidable for various reasons, one of which

is that many of the gauges have been placed at, or near, the sites of promising storage sites.

When dealing with large catchments, the shape of the area is of importance, and when we come to consider the intensity of run-off this factor is of very great importance. A long, narrow drainage area tends to produce a more uniform run-off than one in which the transverse axis is greater than the longitudinal axis.

The gradient of a river is generally steep in its upper reaches, moderate in the middle, and flat nearer the mouth, and, when water has to travel long distances through a dry zone, a large volume is absorbed in the bed and banks and may be all evaporated.

When a catchment area is of large size, and contains extensive areas of entirely different topography, or different climatic conditions, it frequently becomes necessary to divide it up into distinct zones and to calculate the run-off for each separately.

There are usually three classes of run-off calculations. In the first class we are fortunate in possessing accurate and continuous run-off gaugings over a number of years, and also a long and detailed rainfall record for a considerable number of places in the catchment. To be entirely satisfactory the record of stream flow gaugings should be a long one. Unfortunately, such are not available in the Union, the longest extending now over a period of seventeen years only, and those happen to be in the Transvaal, where conditions are slightly more stable than in the case in the more arid portions of the Union.

The longest Cape and Free State records extend only over a period of ten years. This period is far too short, when we look at the fluctuations of the rainfall record over the past forty years. Nevertheless, in these cases we have at least some irrefutable evidence to go on.

The task in such cases is to estimate what the yearly run-off would have been for the period of the rainfall record prior to the commencement of stream gaugings, and for this purpose it may be necessary in many cases to extend individual rainfall records backwards, in the manner described in the second chapter. Having compiled a run-off record on these lines for a long period an endeavour must be made to forecast the run-off in the future.

In the second class no gaugings of stream flow exist at all, but the conditions in the catchment are similar to those of areas

for which information exists, and there are a fair number of rainfall records of long standing, and well distributed over the catchment. This type of case is frequently met with. In such instances the probable run-off, both in the past and in the future, must be deduced from indirect evidence.

In the third class, which in the case of smaller areas is the

TABLE X

Showing the Annual Percentage Run-offs from Catchments gauged by the Irrigation Department.

Name of Catchment	TYPE II CATCHMENT											
	Gamka*		Traka		Seacow		Tarka		Sundays		Riet	
Province	Cape		Cape		Cape		Cape		Cape		O.F.S.	
Area in Sq. Miles	4,055		1,159		1,644		1,885		6,000		4,542	
Description of Catchment	High and Steep Escarpment of Nieuwveld Range and Arid Karroo		Mountainous and Arid Karroo		Mountainous Karroo		Hilly with Extensive Flats—typical Karroo		Ranging from Rough High Mountainous to Flat Country Karroo		Typical Southern Free State Karroo	
	Inches	Per cent	Inches	Per cent	Inches	Per cent	Inches	Per cent	Inches	Per cent	Inches	Per cent
1904–5 ..	—	—	—	—	—	—	—	—	—	—	—	—
1905–6 ..	—	—	—	—	—	—	—	—	—	—	—	—
1906–7 ..	—	—	—	—	—	—	—	—	—	—	—	—
1907–8 ..	—	—	—	—	—	—	—	—	—	—	—	—
1908–9 ..	—	—	—	—	—	—	—	—	—	—	—	—
1909–10 ..	—	—	—	—	—	—	—	—	—	—	—	—
1910–11 ..	—	—	—	—	8·35	1·68	—	—	—	—	10·89	1·82
1911–12 ..	—	—	—	—	14·57	1·31	—	—	—	—	8·38	2·39
1912–13 ..	7·59	4·15	8·67	5·85	9·32	1·18	—	—	—	—	18·22	6·92
1913–14 ..	6·79	2·06	8·00	1·88	7·44	0·27	12·81	5·08	—	—	10·99	1·73
1914–15 ..	4·08	0·98	4·62	2·38	—	—	12·90	4·11	—	—	16·73	3·65
1915–16 ..	3·62	0·83	6·03	5·97	—	—	18·59	4·57	—	—	16·25	0·98
1916–17 ..	9·33	2·14	—	—	—	—	19·34	4·71	—	—	8·06	0·80
1917–18 ..	6·62	4·83	—	—	—	—	8·59	2·68	5·57	1·21	18·39	10·44
1918–19 ..	4·25	4·00	—	—	—	—	14·56	3·91	11·44	5·20	13·59	1·62
1919–20 ..	5·02	—	—	—	—	—	13·75	4·22	14·03	1·75		
1920–21 ..	11·54	—										

* Weir destroyed by flood 1919.

TABLE X (*continued*)

Name of Catchment	TYPE III CATCHMENT				TYPE IV CATCHMENT											
	Stormberg		Valsch		Wilge		Klip		Great Olifants		Bronkhorstspruit		Vaal (Standerton)		Komati	
Province	Cape		O.F.S.		O.F.S.		O.F.S. and Transvaal		Transvaal		Transvaal		Transvaal		Transvaal	
Area in Sq. Miles	970		2,230		5,816		1,616		1,501		612		3,293		2,092	
Description of Catchment	Mountainous Karroo and Grass Veld		Undulating or Flat Country, Grass Veld		Hilly, Undulating and Flat. Grass Veld		Mountainous, Undulating and Flat. Grass Veld		Hilly and Flat High Veld. Grass Veld		Undulating Grass Covered High Veld		Undulating High Veld. Grass Covered		Undulating High Veld. Grass Covered	
	Inches	Per cent	Inches	Per cent	Inches	Per cent	Inches	Per cent	Inches	Per cent	Inches	Per cent	Inches	Per cent	Inches	Per cent
1904–5	—	—	—	—	—	—	—	—	21·56	0·85	20·78	1·18	29·63	7·14	—	—
1905–6	—	—	—	—	—	—	—	—	20·06	0·47	22·04	0·81	22·79	4·62	—	—
1906–7	—	—	—	—	—	—	38·15	15·85	33·11	6·11	29·60	3·36	35·23	14·25	—	—
1907–8	—	—	—	—	—	—	29·32	12·78	23·95	1·60	21·42	2·00	27·79	7·56	—	—
1908–9	—	—	—	—	—	—	38·49	25·27	37·45	10·98	39·57	13·36	34·20	17·38	—	—
1909–10	—	—	—	—	—	—	25·27	13·76	25·28	4·16	25·25	3·69	29·98	13·58	29·22	14·03
1910–11	—	—	—	—	—	—	32·54	9·54	30·58	4·47	28·08	1·84	33·37	13·58	33·25	9·57
1911–12	—	—	—	—	—	—	27·85	8·24	26·17	1·02	21·49	0·61	25·39	5·02	31·28	10·06
1912–13	22·98	4·05	—	—	—	—	25·34	5·42	22·40	1·31	22·59	0·82	28·57	3·48	31·65	10·21
1913–14	14·03	3·42	15·85	4·10	—	—	17·16	1·52	22·69	0·41	21·75	0·40	25·00	1·84	30·17	9·26
1914–15	19·36	4·34	23·88	12·14	31·05	11·21	31·97	10·82	26·37	4·93	39·62	9·34	29·72	7·60	40·60	18·92
1915–16	15·44	1·36	19·90	5·43	26·59	8·05	26·65	14·26	21·21	3·91	18·34	4·09	26·62	7·73	25·19	12·11
1916–17	23·45	4·52	24·48	3·10	28·54	5·08	32·71	5·11	28·63	1·57	26·62	2·48	33·60	5·29	38·49	10·65
1917–18	21·56	6·26	29·69	10·37	41·02	27·28	40·49	38·31	40·11	18·60	38·51	16·31	40·26	29·58	47·58	41·32
1918–19	9·20	0·87	17·33	3·92	22·22	7·52	23·90	8·33	25·02	4·60	23·34	8·01	25·80	7·29	30·85	15·36
1919–20	18·17	5·34	20·52	6·74	22·18	4·47	24·55	3·46	24·78	1·13	20·99	2·14	27·81	4·28	34·88	11·34
1920–21	15·09	4·18	20·36	2·95	25·36	3·75	30·15	6·00	32·35	4·61	30·29	8·15	30·14	9·29	—	—

TABLE X (*continued*)

Name of Catchment	TYPES IV & V INTERMEDIATE				TYPE V CATCHMENT				TYPE VI CATCHMENT					
	Crocodile		Pienaars		Hex		Plat		Hennops*		Schoonspruit†		Klein Marico†	
Province	Transvaal (Central)		Transvaal		Transvaal		Transvaal		Transvaal		Transvaal		Transvaal	
Area in Sq. Miles	797		417		198		51		320		2,233		594	
Description of Catchment	Upper Part Steep Undulating ‡ Dolomite Grass Veld		Generally Undulating Grass Veld		Hilly Country Grass Veld or Rocky		Mountainous—Mainly Grass Veld		Gentle Slopes		Generally Flat		Partly Hilly, partly Flat	
	Inches	Per cent	Inches	Per cent	Inches	Per cent	Inches	Per cent	Inches	Per cent	Inches	Per cent	Inches	Per cent
1904–5 ..	20·20	8·64	21·95	2·34	18·01	4·12	17·66	2·47	22·18	0·89	—	—	—	—
1905–6 ..	20·69	2·49	22·54	2·12	22·65	2·84	23·30	2·02	18·70	0·21	21·62	0·34	—	—
1906–7 ..	30·99	5·08	30·44	4·86	30·61	3·32	22·62	4·13	29·24	1·55	27·47	1·73	29·30	0·68
1907–8 ..	21·16	2·43	20·79	3·43	23·62	1·62	—	—	20·58	0·26	22·79	0·08	18·25	0·21
1908–9 ..	43·05	25·44	40·90	14·28	40·45	10·93	—	—	45·38	6·61	33·88	2·15	37·88	3·03
1909–10 ..	28·27	8·96	25·74	3·34	27·28	5·30	—	—	26·35	0·96	17·30	0·56	15·26	0·78
1910–11 ..	26·93	4·78	25·88	2·45	29·43	2·08	—	—	27·62	0·87	21·30	0·60	25·25	1·13
1911–12 ..	20·33	4·44	20·17	2·02	23·42	1·92	—	—	21·11	0·39	20·18	0·42	22·02	0·19
1912–13 ..	21·32	3·52	23·09	1·54	22·60	4·02	—	—	—	—	19·33	0·87	20·55	0·47
1913–14 ..	18·63	2·52	19·97	0·70	16·15	1·69	—	—	21·84	0·32	15·94	0·54	16·10	0·12
1914–15 ..	42·55	14·92	41·21	12·42	37·08	13·94	—	—	40·74	4·91	27·48	2·00	29·94	2·71
1915–16 ..	22·68	6·61	20·15	2·67	20·73	2·56	—	—	19·40	2·42	19·10	0·26	17·93	0·06
1916–17 ..	29·59	4·66	26·86	2·71	23·46	1·92	—	—	27·15	1·18	23·37	0·56	23·91	0·50
1917–18 ..	48·21	35·44	40·76	12·34	31·79	5·95	—	—	41·02	8·82	30·34	1·94	30·32	0·89
1918–19 ..	25·69	12·69	24·09	6·48	25·54	4·31	—	—	22·80	3·33	21·45	1·17	19·85	0·81
1919–20 ..	22·22	4·37	22·15	1·99	18·44	0·76	—	—	21·70	1·24	18·90	0·48	18·70	0·37
1920–21 ..	28·72	5·84	30·63	6·86	27·25	5·10	—	—	30·76	3·12	23·30	1·37	22·92	0·74

* The surface area of catchment is less than the subterranean catchment.
† The surface area of catchment is less than the total contributing catchment.

TABLE X (*continued*)

Name of Catchment	LARGELY TYPE VI, BUT FED BY POWERFUL SPRINGS						MOUNTAINOUS						TYPE VII			
	Magalies		Aapies		Mooi*		Nels		Groot		Kamanassie		Blijde		White	
Province	Transvaal		Transvaal		Transvaal		Cape		Cape		Cape		Transvaal		Transvaal	
Area in Sq. Miles	424		56		816		77		168		606		200		42	
Description of Catchment	Dolomite Portion Flat, rest Steep, Large Springs at Source		Hilly and Town Area. Large Springs at Source		Gentle Slopes. Large Springs		Steep and Bare Mountainous Country		Steep, Bare, Mountainous Country		Rough, Mountainous Country		Mountainous and Wet		Mountainous and Steep. Grass Veld	
	Inches	Per cent	Inches	Per cent	Inches	Per cent	Inches	Per cent	Inches	Per cent	Inches	Per cent	Inches	Per cent	Inches	Per cent
1904–5	19·42	8·14	—	—	18·23	2·57	—	—	—	—	—	—	—	—	31·72	18·34
1905–6	22·27	5·02	14·76	10·18	19·63	2·71	—	—	—	—	—	—	—	—	30·09	19·37
1906–7	29·22	5·05	31·94	7·64	27·25	2·13	—	—	—	—	—	—	—	—	59·28	40·86
1907–8	20·03	3·71	21·61	10·00	21·32	1·91	—	—	—	—	—	—	—	—	28·69	36·92
1908–9	40·61	13·57	47·37	18·40	33·88	2·74	—	—	—	—	—	—	—	—	61·89	44·84
1909–10	28·00	8·73	27·62	13·68	22·61	2·01	—	—	—	—	—	—	—	—	37·75	44·72
1910–11	28·26	4·95	28·05	11·09	25·31	2·05	—	—	—	—	—	—	36·56	37·80	38·05	33·01
1911–12	19·69	4·17	22·16	9·86	22·04	1·89	10·57	17·03	—	—	—	—	35·40	27·74	—	—
1912–13	21·95	3·54	22·92	9·74	21·48	1·87	13·76	17·96	—	—	10·95	8·79	33·26	29·95	—	—
1913–14	17·46	4·07	22·19	9·33	17·14	1·79	14·75	19·52	8·29	3·98	10·24	8·50	35·05	33·44	—	—
1914–15	33·85	18·28	45·59	17·53	33·30	3·99	7·83	23·75	5·75	6·26	8·79	16·15	71·92	55·20	—	—
1915–16	19·59	4·75	23·14	12·58	21·65	2·03	7·10	34·37	10·93	17·02	17·64	42·17	25·90	39·15	—	—
1916–17	24·51	4·20	27·05	9·83	25·03	1·84	14·38	24·55	9·60	6·88	14·63	9·16	50·63	28·94	—	—
1917–18	40·92	13·59	43·63	14·99	32·85	2·44	10·28	18·88	7·22	9·83	10·27	19·38	73·71	58·15	—	—
1918–19	24·38	9·22	25·68	12·42	23·62	3·39	—	—	2·28	7·46	6·48	4·48	30·88	42·49	—	—
1919–20	19·59	4·19	26·26	11·58	19·84	2·97	8·08	6·68	9·23	9·32	10·80	5·93	41·42	41·46	—	—
1920–21	25·23	5·39	31·40	11·11	25·44	2·71	11·03	—	12·71	14·48	15·03	6·92	51·05	38·86	—	—

* Surface area from which storm flow is received is smaller than total contributing catchment.

most common, there are neither stream flow gaugings nor representative rainfall records. In such cases the estimation of run off becomes very difficult, and is at the best approximate only.

Before dealing with the problems falling under these three classes it is necessary to marshal all the run-off data available for the Union. Table X gives the results of most of the available official run-off gaugings up to the end of the hydrographic year 1921. The catchments are arranged in groups according to the classification given earlier in the chapter. Some catchments have had to be placed intermediately between two classes, viz., the Crocodile and Pienaars Rivers.

The Magalies, Aapies, and Mooi River catchments have been placed in a separate group, as they are fed by very powerful dolomite springs, or "eyes," and the gauged discharges cannot really be expressed in terms of percentages of the run-off from their topographical catchments. The same thing applies in a lesser degree to the dolomitic catchments shown in Group VI. Table XI shows some results obtained from reliable non-official sources.

TABLE XI

Showing Run-off from Catchments obtained from Non-Official Sources.

Type of Catchment ..	Mountainous			
Province	Cape		Cape	
Name of Catchment ..	Sand and Palmiet Rivers		Bulk River	
Area in square miles ..	20·41		11·22	
Source of Information ..	W. Ingham, "Journal" S.A. Inst. of Engineers, January 1920			
Description of Catchment ..	Rough and precipitous and much broken quartzitic mountains			
Year	Inches	Per cent	Inches	Per cent
1901	33·12	10·51	33·12	10·73
1902	34·20	16·92	34·20	18·55
1903	34·13	16·78	34·13	15·26
1904	28·55	20·29	31·52	24·92
1905	43·75	38·94	54·04	31·34
1906	34·15	24·98	41·18	24·14

Table XII shows for a number of typical catchments the total annual gauged run-off in millions of cubic feet. These figures are mainly intended to indicate the enormous fluctuations in the annual run-off.

TABLE XII

Annual Volume Discharged by Certain Rivers in Millions of Cubic Feet.

Year	Gamka	Riet	Great Olifants	Vaal (Standerton)	Crocodile	Hex	Kamanassie
1904–05	—	—	640	6,288	3,232	343	—
1905–06	—	—	330	8,045	954	297	—
1906–07	—	—	7,061	38,399	2,917	470	—
1907–08	—	—	1,344	16,071	953	177	—
1908–09	—	—	14,355	45,366	20,280	2,042	—
1909–10	—	—	3,769	31,143	4,691	667	—
1910–11	—	—	4,765	34,873	2,385	282	—
1911–12	709	—	887	9,752	1,672	210	—
1912–13	2,964	2,092	1,025	7,611	1,386	420	1,360
1913–14	1,334	2,128	322	3,621	864	126	1,233
1914–15	409	13,318	4,523	17,313	11,754	2,388	2,006
1915–16	299	2,048	2,889	15,736	2,783	244	10,473
1916–17	1,890	6,384	1,585	13,615	2,555	208	1,885
1917–18	3,052	1,704	26,018	91,117	31,638	871	2,798
1918–19	1,566	684	4,007	14,411	6,029	508	410
1919–20	—	20,292	977	9,118	1,792	63	903
1920–21	—	2,365	5,202	21,410	3,117	642	1,461

Table XIII shows the results of the gaugings of some of the more important dolomite springs, more especially those feeding the rivers in the special group of catchments shown in Table X, and referred to above.

With regard to the official data, continuous records are available, and the discharge over the weirs for any length of time can be obtained. The results have so far been issued in two publications only. The first of these contains all the gaugings available up to the end of the hydrographic year 1910 to 1911 (September 30, 1911). The latest publication gives results from October 1, 1911, to September 30, 1919. In both cases monthly rainfall and run-off totals are given as well as annual figures.

The classification of catchments made in Table X is somewhat arbitrary, as really every catchment is a law unto itself.

TABLE XIII

Mean Discharges in Cusecs of the More Important Dolomite Eyes or Springs.

	Mooi River Series						Great Marico Series								Magalies River	
	Wonderfontein		Bovenste Oog		Gehardminnebron		Grootfontein		Bokkraal		Rhenosterhoek		Rietspruit		Steenkoppjes	
	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.
1906–07	21·64	17·82	20·01	19·01	25·21	25·16	—	—	—	—	—	—	—	—	—	—
1907–08	20 07	17·94	20·05	19·40	24·01	23·92	9·44	8·33	2·84	2·63	2·66	2·56	3·48	3·30	—	—
1908–09	25·52	21·73	22·35	21·11	23·37	23·24	9·87	8·60	2·76	2·66	2·69	2·57	3·41	3·29	14·09	13·79
1909–10	27·25	24·90	23·13	21·10	23·19	23·06	9·56	7·92	2·73	2·57	2·61	2·54	3·30	3·14	16·03	15·89
1910–11	26·31	24·18	21·61	19·45	23·28	23·19	8·30	8·03	2·56	2·52	2·62	2·61	3·14	3·08	15·89	15·68
1911–12	—	23·02	—	14·48	—	23·13	—	—	—	—	—	—	—	—	—	—
1912–13	—	21·61	—	12·84	—	22·84	—	—	—	—	—	—	—	—	—	—
1913–14	—	17·00	—	12·12	—	21·56	—	—	—	—	—	—	—	—	—	—
1914–15	—	—	—	18·47	—	21·51	—	—	—	—	—	—	—	—	—	—
1915–16	—	—	—	15·60	—	21·09	—	—	—	—	—	—	—	—	—	—
1916–17	—	—	—	13·58	—	21·63	—	—	—	—	—	—	—	—	—	—
1917–18	—	—	—	16·69	—	21·24	—	—	—	—	—	—	—	—	—	—
1918–19	—	—	—	20·36	—	22·95	—	—	—	—	—	—	—	—	—	—
1919–20	—	—	—	16·44	—	25·89	—	—	—	—	—	—	—	—	—	—
1920–21	—	—	—	14·39	—	26·24	—	—	—	—	—	—	—	—	—	—

For general practical work, however, some sort of generalization is necessary, and the catchments included in the various groups are considered to be representative. Officially no attempt has as yet been made to classify catchments in this manner.

A study of Table X reveals many apparent anomalies, but the figures show, nevertheless, more consistency than might be expected. In diagrams in Fig. 7 an attempt has been made to

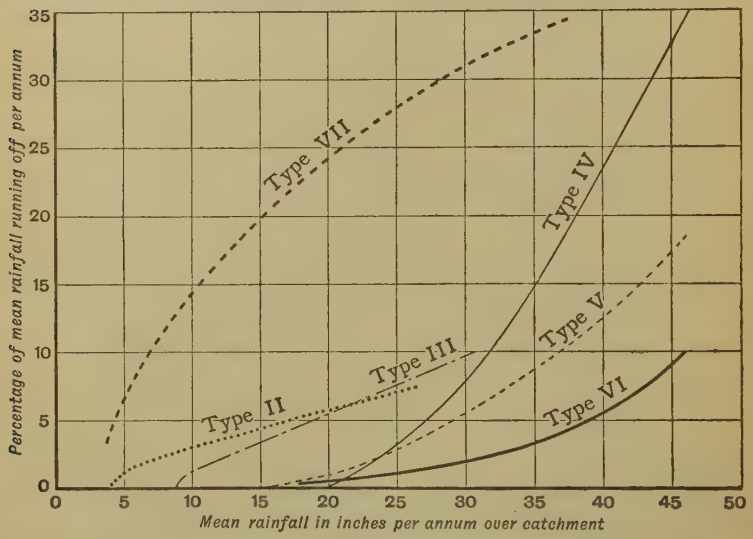


FIG. 7.—Typical annual run-off curves for various catchments in South Africa.

represent the mean relationship between annual rainfall and run-off for each class or type of catchment by a curve. The drawing of such curves requires considerable judgment, and those shown might possibly be improved upon ; but, considering the data available, they may, in the author's opinion, be taken as representative of the different classes of catchment dealt with.

There are very great differences in the value of supporting data for the different curves. Thus the curve for Type IV catchment is based upon seventy-three gauging station years, whereas that for Type III catchment is only supported by thirteen station years. The curve for the mountainous catchments is given with some diffidence. The conditions operating in different catchments of this type are so very diverse that it is

almost impossible to draw a curve which will be representative of catchments so different as those gauged.

With many mountain catchments there is always a great element of doubt as to whether the rainfall as measured in the gauges installed really represents the true mean rainfall. Where such catchments are situated on the escarpments of the great coastal ranges the rainfall varies greatly with altitude and with other topographical features.

The heaviest rainfall will frequently occur in the most inaccessible portions of the catchment where no rain gauges can be erected or observed. Some of the Nels River percentages are, for example, open to doubt from this cause.

Geological and general climatic conditions also have a very great influence on the run-off from catchments of this kind. This particular curve should therefore only be taken as indicating the general character of the run-off percentages as compared with the curves for other types of catchment.

Eliminating mountain catchments, comparison of the different curves shows very clearly the extent to which the hard, bare catchments of the Karroo, as represented by Curves II and III, are better yielders of run-off than those of the Transvaal and Orange Free State with their more favourable climate and more ample covering of soil and vegetation.

From what has been said it will be realized to what a large extent run-off calculations are based upon inadequate data and much guess work. For the time being the South African engineer is helpless in the matter. All that can be said is that the problem to-day is very much lighter than it was, say, ten years ago.

In South Africa hydrographic investigations have been carried just far enough to throw a flood of light on the problem as a whole, and to indicate in what directions new lines of research should be opened up.

Though at first the investigations of the Hydrographic Survey were undertaken mainly with the object of collecting data of a purely general kind, practical considerations very soon forced the Department to undertake elaborate investigations in catchments for particular conservation projects, to the exclusion of the more general investigation of typical catchments. For general purposes the catchments dealt with by the Survey are far too large, and the conditions existing therein much too varied, to serve as a really good basis for the investigations of other catchments.

The aim should now be to work with much smaller areas, each typical in itself. For general application the data obtained from the gauging of rivers with catchments thousands of square miles in extent are of doubtful value.

The different problems will now be dealt with in the order referred to above.

In the case where continuous stream flow gaugings and records of the daily rainfall at a large number of places in the catchment are available, the first thing to do is to establish the relationship, where possible, between a particular rain period, lasting one or more days, and the measured run-off yielded by that rain. This is only possible when we are dealing with intermittent rivers. Where the flow is perennial the problem is complicated by the effect of ground storage. For the present attention will be confined to intermittent rivers, which furnish the most frequent and difficult cases which have to be dealt with in South Africa.

The relation between the individual rainstorm and the measured run-off provides the key to the position. For this purpose the rainfall stations must be carefully placed and well distributed. The daily rainfall at each of the rain gauge stations must be recorded on a comprehensive statement, and from this the mean fall over the catchment for each particular storm or rain period calculated. This calculation of the mean may be made in respect of the whole catchment, or in respect of zones or divisions of the catchment, when the area under consideration is very large and diverse.

For each rain period the total volume of water reaching the ground must then be calculated, which is a matter of simple arithmetic. The volume of measured run-off should now be recorded on the same statement and the dates on which the flow commenced and ceased noted. The analysis will show, in the first place, what mean rainfall over the catchment is necessary to produce any run-off at all.

This critical amount will be found to vary very considerably. It will be large when the rain was preceded by a lengthy period of drought, and small if the surface was wet from recent rains. It will vary with the season of the year, and also according to amount, intensity, and position of individual falls. A mean for the catchment of, say, half an inch may be found to be made up of individual falls ranging from *nil* to over one inch. An isolated fall of one inch over a small area near the gauging weir would

produce a considerable volume of discharge. If the same amount fell over a small area, at a considerable distance from the gauging weir, its influence would be small. All circumstances of this kind must be taken into consideration.

When the statement, or chart, has been properly prepared it will be seen, without much chance of error, which individual falls contributed to produce run-off and which did not, and all falls which obviously did not contribute should be excluded when calculating the catchment mean which yielded the discharge.

In Table XIV are shown a few typical examples of such an analysis by way of illustration only. The analysis should of course, be continuous, showing the condition of the catchment for every day throughout the year. The examples are taken from an analysis actually made for a large catchment which is divided into four zones. These subdivisions are not shown in the table so as not to complicate the example unduly. The letters at the head of the columns represent rain gauge stations distributed over the area.

By carefully tabulating all the results obtained from such an analysis it will be possible to construct a number of run-off curves for the catchment. In making these curves the mean rainfall should be plotted as ordinates, and the percentages running off in each case as abscissæ.

One curve should be constructed for general storms all over the catchment or sub-catchment when the rain lasts from one to, say, three days. Another curve should represent the relationship during extended periods of rainy weather. A third curve should, if possible, be drawn for conditions which will produce very high floods.

If the catchment is very large and the records indicate that storms over distinct portions are frequent, the remainder of the area receiving at the same time little or no rain, then separate curves should be made for such local conditions. This should be done whenever possible, as, when partial rains are received, means calculated for the whole catchment are apt to give misleading results. With a record of, say, five years, the number of individual run-off producing storms is so large that the construction of a number of such curves becomes possible.

If sufficient data exist the curves should be drawn for each class of run-off producing storm or rain period, viz., one for rain falling on a dry catchment and one falling on the same catchment which is already moist or wet from recent rains.

TABLE XIV

Some Examples of an Analysis of Rainfall and Run-off in a Catchment of Large Size.

Date		A	B	C	D	E	F	G
October	28	0·50	—	—	—	—	—	—
1918.	29	0·80	1·50	0·39	0·50	1·25	0·43	0·90
	30	—	—	—	—	—	0·06	—
		1·30	1·50	0·39	0·50	1·25	0·49	0·90
November	1 to							
1918.	30	0·23	0·00	0·11	0·23	0·00	0·00	0·00
December	2	0·20	0·42	0·46	0·50	0·25	0·45	0·25
1918.	3	—	—	—	—	—	—	
	4	—	—	—	—	—	—	
		0·20	0·42	0·46	0·50	0·25	0·45	0·25
February	15	0·23	0·00	0·20	—	—	—	—
1919.	16	0·00	0·08	0·00	0·32	0·00	0·08	0·26
	17	1·27	1·09	1·50	1·21	0·82	1·72	0·76
	18	0·00	0·00	0·00	0·18	2·37	1·68	0·55
	19	—	—	—	—	—	—	—
	20	—	—	—	—	—	—	—
		1·50	1·17	1·70	1·71	3·19	3·48	1·57
May	1							
1919.		—	0·16	0·30	0·67	0·30	0·15	—
December	28	—	0·08	—	—	—	0·12	—
1919.	29	0·10	—	—	—	0·09	—	—
	30	0·10	0·22	0·26	0·18	—	—	0·08
		0·20	0·30	0·26	0·18	0·09	0·12	0·08
November	24	—	0·10	—	—	—	—	—
1920.	25	0·10	0·18	0·27	0·48	0·12	0·50	—
	26	0·22	—	0·60	0·69	0·26	0·69	0·41
	27	—	—	—	—	—	—	—
	28	—	—	—	—	—	—	—
	29	—	—	—	0·15	0·10	0·08	—
		0·32	0·28	0·87	1·32	0·48	1·27	0·41
February								
1921.		2·57	2·09	2·00	3·26	2·50	2·40	1·99
March								
1921.		3·06	4·93	4·15	3·66	3·45	2·02	2·58
December	23	0·13	0·22	—	—	—	0·69	0·25
1921.	24	0·10	—	0·02	—	0·45	0·47	—
	25	0·76	0·26	2·77	0·84	0·25	0·43	—
	26	—	—	0·01	0·04	0·55	0·31	0·80
	27	—	—	0·03	0·14	2·05	1·00	0·50
	28	0·70	1·34	0·96	0·83	2·14	0·81	0·81
	29	—	—	0·06	0·10	—	0·06	—
	30	—	0·10	—	—	—	—	—
	31	—	—	—	—	—	—	—
		1·69	1·92	3·85	1·95	5·44	3·77	2·36

TABLE XIV (*continued*)

Date		H	I	J	K	L	M
October	28	—	—	—	0·54	0·03	—
1918.	29	0·74	0·00	0·63	0·85	0·67	0·75
	30	—	—	—	—	—	0·29
		0·74	0·00	0·63	1·39	0·70	1·04
November 1 to 1918.	30	0·00	0·56	0·00	0·00	0·00	0·07
December	2	—	0·41	0·23	0·23	0·00	0·23
1918.	3	0·35	—	—	—	—	0·04
	4	—	—	—	—	—	—
		0·35	0·41	0·23	0·23	0·00	0·27
February	15	—	—	—	—	—	—
1919.	16	0·00	0·00	0·20	0·25	0·26	0·24
	17	0·62	0·55	0·36	0·44	·024	0·93
	18	0·50	0·45	0·20	0·00	1·21	0·00
	19	—	—	—	—	—	—
	20	—	0·05	0·11	0·41	—	—
		1·12	1·05	0·87	1·10	1·71	1·17
May	1	—	0·25	0·64	0·46	—	—
1919.							
December	28	0·27	0·18	—	—	—	—
1919.	29	0·12	0·02	—	—	—	0·09
	30	0·24	—	—	—	—	0·07
		0·63	0·20	0·00	0·00	0·00	0·16
November	24	—	—	—	—	—	—
1920.	25	0·43	0·68	0·50	0·94	0·56	0·29
	26	0·44	0·33	—	—	0·40	0·42
	27	—	—	—	—	—	—
	28	—	—	—	—	—	—
	29	0·11	0·06	—	—	0·24	0·05
		0·98	1·07	0·50	0·94	1·20	0·76
February 1921.		1·94	1·83	0·27	1·43	N.R.	2·03
March 1921.		2·41	3·20	1·92	1·37	N.R. —	3·16
December	23	0·13	—	—	—	—	—
1921.	24	1·15	0·10	2·14		—	0·82
	25	0·22	0·55	0·14		0·72	0·28
	26	—	1·00	0·82		0·16	0·12
	27	0·16	0·08	0·06		0·40	—
	28	2·20	1·56	3·37		0·24	2·46
	29	—	0·12	0·14	No Record	1·53	0·30
	30	—	—	—		0·31	—
	31	—	—	—		—	—
		3·86	3·41	6·67	—	3·36	3·98

TABLE XIV (*continued*)

Date	N	Mean Rain over Catchment	Gauged Stream Flow Mill. cub. ft.	Period of Flow	Per Cent of Rainfall	Remarks
October 28	—					
1918. 29	0·72			31 Oct.		
30	0·15	0·84	24	& 1 Nov.	0·2	Catchment dry
	0·87					
November 1 to						Scattered
1918. 30	0·11	0·09	*Nil*	—	0	showers during month
December 2	0·27					
1918. 3	0·03	0·31	*Nil*	—	0	Catchment dry
4	—					
	0·30					
February 15	—					
1919. 16	0·06					
17	0·64			18 to		
18	0·40		428·454	24 Feb.	1·9	Catchment dry
19	—					
20	—					
	1·10	1·60				
May 1	0·33	0·23	71·550	1 to 3 May		Catchment dry. Origin of freshet unknown
December 28	0·10		61·934	28 to 30		Catchment dry.
1919. 29	0·01					Origin of freshet
30	0·32					unknown
	0·43	0·19				
November 24	—					
1920. 25	0·34					
26	0·57		*Nil*	—	—	Catchment dry. All run-off absorbed
27	—					
28	—					
29	0·37					
	1·28	0·83				
February 1921.	1·78	2·00	604·646	15 to 24	2·2	Rain fell on 14 days
March 1921.	3·13	3·00	617·0	4-10, 17-27, 30 Mar. & 1 Apl,	1·4	General rain which fell on 20 days
December 23	—					
1921. 24						
25						
26						Catchment dry
27						for 14 days
28		3·52	13,055·5	24 to 31	26·0	prior to
29	No Record					storm. This
30						rain produced
31						a record flood

For each curve the starting point will be found to be different ; and in the case of the two curves for the same class of rain period the minimum mean rain over the catchment producing run-off will be much smaller on a wet catchment than on a dry one. The difference between the wet and dry catchment curves is considerable only in the case of comparatively low rainfalls. As the mean rainfall increases the curves approach one another, and after it exceeds four to five inches they generally merge into one curve.

In the case of dry catchments, which is the usual condition, it rarely happens that less than one-half inch of mean rainfall will produce any appreciable run-off. When the catchments are large, that is, measuring over, say, 1,000 square miles, and especially if irrigation is practised within the area, the minimum mean rain-producing stream flow is generally much higher in the case of the usual class of storm lasting one to three days with no high intensities. In the Sundays River catchment, for example, above the Lake Mentz Reservoir, with a catchment of over 6,000 square miles, a mean rainfall of over one inch during a rain period is necessary to produce appreciable stream flow. There are at present no large irrigation works in the catchment, but there are numerous farm irrigation schemes diverting flood water which, in the aggregate, dispose of a very considerable percentage of the smaller floods. Irrigation of this kind is so common on all the rivers in the Karroo that in the present stages of development run-off percentages must be taken to represent the net stream flow, allowing for the diversion of a varying percentage of water by farmers for irrigation. All the official gaugings of stream flow show net discharges, no attempt being made to estimate the water diverted by irrigation furrows.

Some typical run-off curves for individual storms are shown in the annexed diagram (Fig. 8). The curves for the upper part of the catchment, which is mountainous and far away from the gauging site, show a much lower rate of run-off than is the case with another section of the catchment which, though arid and much flatter, is much nearer the gauging weir and reflects the influence of local storms.

The third curve shows the run-off percentages from the whole catchment during lengthy periods of wet weather, generally with rainfall of low intensity. After several days of such rain the catchment surface becomes wet and the rate of run-off increases rapidly.

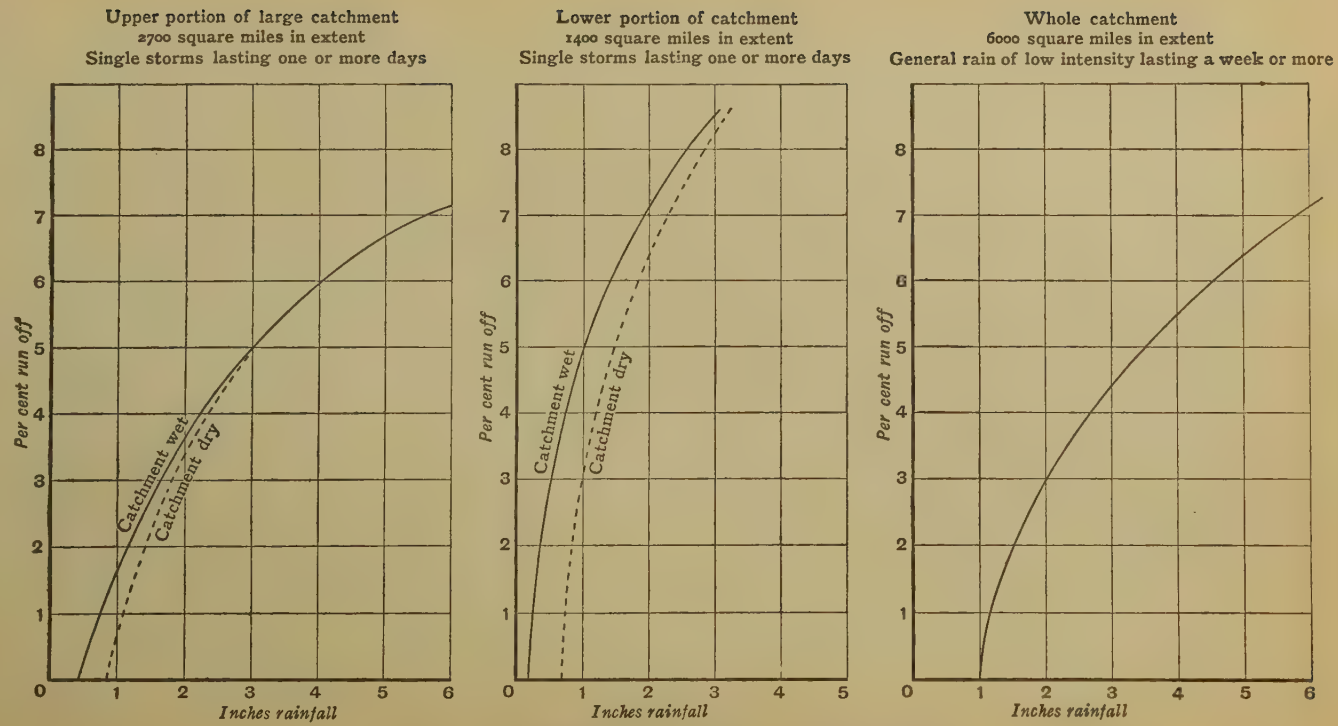


FIG. 8.—Typical run-off curves for individual storms.

Similar curves to those shown were prepared by the Irrigation Department of the Union of South Africa for a portion of the Tarka River catchment (1,885 square miles) and a portion of the Stormberg catchment (970 square miles) and published in the Annual Report of the Director of Irrigation for the year 1917 to 1918.

Both catchments are gauged by the Hydrographic Survey Branch, and are provided with a large number of carefully distributed rain gauges.

These curves represent mean conditions, and no account was taken of the state of the catchment prior to each rain period. In Table No. XV the results of these analyses are given.

TABLE XV

Showing Mean Percentages of Run-off of Individual Rainfall Periods for Certain Catchments.

Mean Rainfall over Catchment in Inches	Tarka Catchment 1,885 square miles	Stormberg Catchment 970 square miles
	Percentage Run-off	
0·50	2·0	1·4
0·60	3·0	2·2
0·70	3·7	2·9
0·80	4·0	3·1
0·90	4·7	3·9
1·00	5·1	4·4
1·20	5·8	5·1
1·40	6·4	5·8
1·60	7·0	6·5
1·80	7·4	7·1
2·00	7·7	7·8
2·20	8·0	8·4
2·40	8·2	8·9
2·60	8·4	9·5
2·80	8·6	—
3·00	8·7	—

(*Taken from Report of Director of Irrigation for Union of South Africa,* 1917 *to* 1918, *page* 77.)

It will be noticed that the percentages for the Tarka catchment correspond closely to those represented by the wet catchment curve of the 1,400 square mile catchment shown in the diagram. The percentages in the case of the Stormberg catchment are smaller than those for the Tarka areas for low rainfalls, but are higher for mean rainfalls above two inches. The reason for this is that the catchment is more absorbent and better clothed with vegetation owing to a better rainfall. Once it becomes saturated, however, the percentage of run-off is higher.

The analysis should be extended also to embrace the entire year. This annual computation will show what proportion of the rainfall contributed nothing to the stream flow passing the gauging site. The total volume of stream flow gauged should be expressed as the percentage of the total mean precipitation for the year, and also of that portion of the mean rainfall which has been shown to contribute to the stream flow.

Armed with the curves for the individual storms, and the percentages calculated for the year, it now becomes necessary to analyse the rainfall over the catchment for the whole period for which rain statistics exist prior to the date on which stream flow gaugings were commenced.

Here one is at once faced with a shortage of rainfall records, as the large number on which the first analysis was made was, for the most part, composed of special gauges established by the Hydrographic Survey Department when the river gauge was erected. The first task, therefore, is to select from the old established rain gauges of the Meteorological Department a number of representative gauges, as described in the chapter on rainfall. As the length of some of the records may be shorter than others, these shorter records must be extended backwards, as already described.

To the available limit for which daily rainfalls are obtainable the analysis should be made in the same way as was done for the stream flow gauging period, and the curves already obtained should be applied to compile a statement of the estimated run-off for each year.

It will generally be found when dealing with large catchments and a small number of, or a bad distribution of, rain gauges that a certain volume of stream flow cannot be traced to any recorded rainfall. This is due to heavy local showers which escape being recorded by any of the established rain

gauges. The procedure adopted tends, therefore, to underestimate the annual run-off. Allowance must be made for the stream flow which cannot be correlated to recorded rainfalls. The proportion of this uncorrelated rainfall is greater in years of bad rainfall than in good years, and greater in the case of large catchments than in small ones. A percentage correction is therefore sometimes advisable, which may range from about twenty-five per cent in years of bad run-off to nothing in years of good run-off.

When this analysis has been carried back to the stage where sufficient daily data are not available, the best use possible must be made of annual totals, both derived from the actual gaugings or as estimated from the daily falls. The investigator should be now in possession of the actual or estimated total run-off for each year for the past thirty or forty years.

A forecast of the future can only be made by constructing a probability curve which will indicate the percentage frequency with which various amounts of annual run-off may be expected. The above-described procedure is the best that can be done with the most favourable data at our disposal. It is a very tedious operation, but when big issues are involved it should nevertheless be conscientiously done.

We have not yet succeeded in deriving a rational expression for the relation between run-off and rain, even in respect of an individual storm, let alone one which applies to a full year. Such a formula for an individual storm should be a function of total rain recorded, intensity of the fall, and previous condition of the surface of the drainage area, and of the bed of the stream, and, finally, of the prevailing weather conditions, such as temperature and wind, as both of these greatly affect the evaporation from the surface.

When we are dealing with seasonal totals the introduction of most of the above-named factors is clearly impossible, though it might be feasible to introduce a "drought factor," which would give some indication of the average state of the surface during the rainy seasons, and also an "intensity factor," based upon the mean individual fall for the year, obtained by dividing the aggregate of all the individual falls for the year by the aggregate of the number of rain days for the year.

We now come to the second class of run-off calculations. To deal satisfactorily with areas in this class the engineer must have sufficient knowledge of the conditions obtaining in the

catchment to enable him to compare it with other and generally similar catchments belonging to the first class, in respect of which run-off curves can be, or have been, prepared in the manner already described. When dealing with large catchments it is rarely the case that the physical or climatic conditions are identical, and modifications of the curves must be made of a somewhat arbitrary character. Such modifications may be effected by compounding the curves of two similar catchments when it is assumed that the catchment to be studied presents conditions more or less intermediate between those of the catchments with which it is compared.

Alternately the curves for one catchment may be modified by making proportionate corrections. In either case the accuracy of the estimate will depend wholly on the judgment of the investigator.

Immediately it is decided to estimate the run-off from a catchment of this class steps should be taken to measure the run-off and also to collect any data available relating to the number of floods, the size of floods, etc. All such information will be of service as a check on the calculations.

Having determined what run-off curves are to be employed, the subsequent procedure will be identical to that previously described for the first class of case considered.

The third class of case is the most difficult one to deal with, as there are no daily rainfall records available, and any attempt to build up the total run-off for the year as described in the previous cases is impossible, unless the catchment is very small and rainfall records for stations only a short distance from its boundaries are available. In these cases two alternative methods may be adopted. Firstly, and provided sufficient data are available, one may fall back on the monthly and annual rainfall totals of the nearest rain gauges available, and deduce therefrom the monthly rainfall over the catchment for each month over as long a period as possible.

It will be further necessary to analyse the rainfall record of the stations selected so as to form some definite idea as to the general character of the rainfall during each month, and classify it according to whether the monthly total was composed of one or more moderate storms, or of a number of light showers, or of one or more exceptionally heavy storms. Probably four classes will suffice according to whether the rainfall, from a run-off yielding point of view was, say, very good, good, moderate, or bad.

A similar analysis should then be made on a monthly basis in respect of the most similar catchment for which actual data exist, and approximate run-off curves prepared for each of the four classes. These must then be applied to the catchment area under investigation.

Secondly, if the rainfall data are insufficient to justify the adoption of the method just described it becomes necessary to fall back upon the best available estimate of the mean rainfall over the catchment for each year, and then to apply the annual run-off percentages taken from the annual run-off curves for typical catchments given earlier in the chapter. Such computations will be approximate only. They will, nevertheless, be much more accurate than one based upon arbitrary percentages of mean annual rainfall and run-off.

So far only intermittent streams have been dealt with of the type met with over the greater part of the Karroo in the Cape and Free State Provinces. These rivers form the sources of the majority of the larger irrigation works in South Africa, and from a hydrographic point of view present by far the most difficult problem. There remain other very large areas in the northern Free State and the Transvaal which must be dealt with.

These areas enjoy a much better rainfall with higher annual minima. The country is, on the whole, better covered with soil than is the case in the Karroo, and is consequently much better clothed with grasses and other vegetation. Added to this the plateau area is mainly very flat though intersected by several great escarpments and ranges, such as the Witwatersrand, the Magaliesberg, and others.

The rivers in this area are mostly perennial, though during the late winter and spring months the discharges often dwindle to very small dimensions. The curves of mean percentage run-off for Transvaal rivers are wholly different in form to those of the Karroo, as may be seen from the diagrams on page 84 in this chapter.

The starting point of the Transvaal curves is between fifteen and twenty inches mean rainfall per annum, compared with from five to ten inches in the Karroo, which gives some indication of the large amount of absorption in Transvaal catchments.

In the case of rivers rising on the Witwatersrand, Drakensberg, and other high ranges the annual run-off percentage rises very rapidly for annual rainfalls of over twenty inches. It is

approximately 3·5 per cent from twenty-five inches, 7·5 per cent from thirty inches, 15 per cent from thirty-five inches, and 28 per cent from forty inches.

In the case of most of the rivers of the Transvaal, however, the conditions are much less favourable to high percentages of run-off, and with annual rainfalls up to twenty-five inches the total run-off rarely exceeds four per cent and is generally below two per cent.

In the northern parts of the Transvaal the run-off expressed as a percentage of the annual rainfall is probably very low in spite of good rainfall. The country is flat, sandy and generally absorbent.

In the western Transvaal the run-off is very small owing to the sandy nature of the surface, and unless rain falls with considerable intensity very little of it reaches the streams. Thus, for example, during the seasonal year 1916-17, the catchment of the Klipdrift reservoir in the Potchefstroom district yielded only one-half per cent of a rainfall of approximately twenty-one inches, and in 1918–19 the yield was only two per cent of thirty inches, the catchment area being 330 square miles.

On the other hand in 1917-18 the run-off was approximately five per cent of thirty-one inches of rainfall. Generally western Transvaal catchments yield from one-half per cent to two per cent of the total annual rainfall only.

The Transvaal catchments hitherto dealt with are all within geological zones other than in the dolomite. In that province particularly the geological structure of the country has a predominating effect on the run-off conditions.

The differences in the absorptive power of different formations are enormous. A large river traversing granite country for long distances will show a steady decreasing discharge as one progresses downstream. The Karroo beds, composed of shales and sandstones, are very much less absorbent, and Karroo catchments are therefore much better yielders of run-off.

The dolomite areas of the Transvaal and Bechuanaland are, however, quite exceptional in this connexion, and these areas must therefore be dealt with separately.

There are three main dolomite areas in the Union:

Firstly, in the north-eastern Transvaal, having an area of approximately 4,000 square miles and a mean rainfall of thirty inches, and a mean run-off percentage of two.

Secondly, in the south-western Transvaal, having an area

of 15,000 square miles and a mean rainfall of twenty-three inches, and a mean run-off of one per cent.

Thirdly, in Bechuanaland, having an area of about 16,000 square miles, a mean rainfall of about twenty inches and no run-off at all.

The dolomite is a largely developed geological formation of great age and is a magnesian limestone. Apart from the areas already mentioned, there is a narrow strip of this formation outcropping along the foot of the Zwartbergen in the division of Oudtshoorn in which are situated the famous Cango caves. This outcrop, no doubt, affects the run-off of many streams which traverse it in their courses from the Zwartberg slopes to the Olifants River, to which they are all tributary, but owing to its narrow width the effect is comparatively small and the area will be ignored for the present as no run-off data bearing on the subject are available.

In the dolomite areas of the Transvaal and Bechuanaland the rock is either exposed or only lightly covered by red sandy soils, and the surfaces are generally flat. The rock is to some extent soluble, and in the course of ages the surface water penetrating it has dissolved large quantities of it, so that the formation is now intersected with a network of fissures, varying from hair cracks to large underground channels. Enormous caves or caverns are very common. Occasionally the surface breaks through into these caverns forming sink-holes of great depth, such as the Wondergat near Mafeking or the Otjikoto Lake in the north of the South-West Protectorate. Rivers in the dolomite area occasionally disappear into a sink- or swallow-hole, to emerge again many miles away. Many of the most permanent rivers in the Transvaal and Bechuanaland originate in large springs, or eyes, which may yield permanent flows of forty cusecs or more. Amongst the permanent rivers arising in this manner are the Mooi, Schoonspruit, Marico, Aapies, Hennops, Klip, Molopo, Kuruman, and others. It is only natural that under such conditions the run-off from catchments containing a large extent of dolomite should be small when measured as stream flow in the river draining the area. Water percolating into the subsoil of such a catchment may appear in an "eye" many miles away and form the source of a river in another topographical catchment altogether. So uncertain are we regarding the progress of underground water in this formation that it has been specially dealt with by the

legislature and many drastic restrictions are placed on the use thereof.

A considerable number of dolomite rivers are under observation by the Hydrographic Survey Branch, from which it is seen that the percentage of the rainfall which appears as stream flow is very small, and it is clear that very great volumes of water falling on the surface are permanently stored underground or find their way into ungauged rivers.

On the Schoonspruit, for example, with a catchment area of 2,233 square miles, it is very rare that a mean rainfall of twenty-five inches will yield more than one per cent run-off. The biggest percentage run-off on record since 1904 was 2·15 from a rainfall of thirty-four inches. In these rivers the permanent flow from springs may amount to as much as two-thirds of the total flow for the year, so that surface drainage contributes only a comparatively small volume to the total annual flow.

In our present state of knowledge it is hardly possible to carry the analysis of rainfall and run-off in the Transvaal very much further than is given in the data published by the Irrigation Department, and shown in an abbreviated form on Table X in this chapter, and in the mean percentage run-off curves drawn by the author as representative of typical classes of catchment.

The daily and monthly discharges of all the gauged rivers are available, but owing to the large amount of absorption and of ground storage, which gives most Transvaal rivers a perennial flow, the correlation of individual storms or of monthly rain totals and the run-off for corresponding periods is impracticable. Even when working with annual figures the relationship becomes occasionally confused owing to the large carry-over of ground storage from one year into the next. If Table X is examined it will be seen that in the case of Type IV catchment, for example, years of heavy rainfall, such as 1908 to 1909 and 1917 to 1918, cause the percentage of run-off during the following years of moderate rainfall to be much inflated.

Fortunately the Transvaal is now fairly well provided with hydrographic gauging stations of sufficient standing to give a much better idea of the run-off conditions over the greater part of the country than in the Karroo, and for the time being the engineer must rest content with the data available in the Hydrographic Survey Office or with the mean run-off curves shown in this chapter.

Natal has been somewhat neglected from a hydrographic point of view, mainly owing to the fact that irrigation is for the present very little practised in that province.

No official stream flow gaugings are available, and rough approximations can only be arrived at by applying percentages obtained from the most similar Transvaal catchments.

In later years, when data over a longer period are available, a commencement should be made to express the variations in the annual run-off, and more especially the maximum and minimum yearly run-off by means of probability curves.

A valuable contribution to this subject is contained in a paper entitled the "Probable Variations in Yearly Run-off," by L. Standish Hall, in the Am. Soc. of C.E., Vol. LXXXIV, 1921, and in the discussion thereon.

The purpose of Hall's paper was to study the actual variations of measured yearly run-off of Californian rivers in order to construct a probability cross section paper on which by means of a straight line relation, the probable minimum and maximum yearly run-offs, which are likely to occur once in a hundred years, may be determined for any stream from the available run-off data for that stream. While the general method of treatment of the subject as dealt with in the paper is, in the author's opinion, sound, exception must be taken to the data used for the purpose. In the first place, the records are much too short, and in the second place the natures of the catchments are too diverse. The catchments used by Hall vary in size from 30 to 225,000 square miles. It would appear, in South Africa at any rate, to be useless to attempt such a treatment of the subject unless the catchments are more or less comparable as regards size and physical character, and are subject to tolerably uniform climatic conditions. Furthermore, the study of rainfall records in South Africa, and especially in the more arid regions thereof, shows the existence of long periods of excess and deficiency of rainfall, as was pointed out in Chapter II. The record period should therefore be long enough to ensure that the investigations are based upon average rainfall conditions, and are not representative of an abnormal period of either excess or deficiency.

In the Transvaal, where climatic conditions are more stable than elsewhere in the Union, precise run-off data extending over periods of twenty years will soon be available, and analyses of the kind proposed by Hall can then be made with advantage.

CHAPTER IV

HIGH FLOODS. DISPOSAL OF SURPLUS WATER AND PROTECTION AGAINST FLOODS

PRACTICALLY in every irrigation scheme the engineer is faced with the problem of dealing with high floods in such a manner that the safety of the works themselves and of cultivated lands, dwellings, to say nothing of lives, shall be ensured. Railway and road engineers have to deal with the same problem, more especially when designing bridges and works for dealing with cross drainage. In the case of storage dam or weir construction the matter is of supreme importance, and in the case of earth dams it may be said to be the most vital factor in the design. No factor is less appreciated by amateur dam builders, and even by inexperienced engineers. Hardly a year passes without leaving its record of some disaster of magnitude caused by insufficient attention to details of the design of works for the disposal of great floods, or to the fact that seemingly adequate estimates of high flood discharge have been greatly exceeded. In the case of farmers' storage dams in South Africa the almost universal failure to provide sufficient spillway capacity is notorious. These troubles are due to the great uncertainty regarding the greatest intensity of flood discharge to be provided for.

As was shown in the chapter on run-off, the conditions which give rise to a flood are a big rainfall of high intensity all over the catchment. The rate of run-off will be increased if the surface of the catchment is already wet, or if the storm, falling on a dry surface, continues long enough to saturate the ground thoroughly. The shape of a catchment is likewise an important factor. Thus a very long and narrow catchment will tend to produce a lower rate of discharge than a broad and comparatively short catchment. On small compact catchments the rate of flood discharge is invariably greater than from large areas In the first case the floods are of high intensity, but of short duration, whereas

in the second case they are less violent but of much longer duration. On great rivers with extensive tributary systems, whose respective catchments are far apart, it rarely happens, fortunately, that at the junction of any two main tributaries both have attained their maximum rate of discharge at the same time.

There is, however, nothing to prevent such a contingency, which would depend upon the direction of travel of the storm, and the probability of this happening must be taken into account even if no actual record of such a catastrophe exists.

As was pointed out when dealing with rainfall, whereas the minimum precipitation for the year can never be less than *nil*, the maximum precipitation for the year cannot be predicted. Thus, for example, the total rainfall at Queenstown for the year 1891 was nearly ninety per cent above the mean for forty-nine years, and was twenty-six per cent greater than the next highest total. With individual falls the relation of maxima to the mean is often even more exaggerated. Record downpours are frequently very local, but occasionally a storm of universal severity will affect an enormously large area.

Such a storm broke over the southern part of the province of the Cape of Good Hope during the first week of May, 1916, (a most unusual time of the year for such an occurrence). This affected the greater part of three very large river systems and caused record floods in each of them, one being of a truly devastating character. In the latter case, viz., the Gamtoos River, the damage done would have been very much worse had the two main feeders of the river reached their maximum flood at the same time.

A few of the daily rainfalls in the area in question during the course of this particular storm are shown in Table XVI.

In a comparatively new country such as South Africa reliable records of floods cover a comparatively short period of time, and even traditional records only go back one hundred years or less so far as nine-tenths of the area comprising the Union is concerned. When dealing with flood records we are, in South Africa, faced with a further trouble. Owing to the effects of white settlement during the past hundred years the surface conditions of the country, especially in the drier parts, have radically altered.

Prior to the days of European settlement the main lines of drainage in the Karroo, for example, were much less clearly

TABLE XVI

Some Daily Rainfalls in Southern Cape Midlands during the Abnormally Widespread Storm of the 3rd and 4th May, 1916, which produced Devastating Floods.

Name of Station	Max. Daily Rainfall in Inches	Average Annual Rainfall in Inches
Doorn River ..	9·10	25·72
Molen River ..	18·23	21·07
Ruigte Vlei ..	9·75	28·14
Sunnyside ..	13·50	19·92
Sourflats ..	14·10	34·64
Buffels Klip ..	7·10	7·42
Wolve Kraal ..	8·75	16·23
Dengas ..	9·05	11·37
de Hoop ..	16·00	13·93
Welgelegen ..	17·61	19·91
Vrolijkheid ..	8·15	6·22
Roode Els Bosch	9·15	8·60
Spitzkop ..	5·85	6·27
Middelkraal ..	5·30	6·73
Willowmere ..	5·96	9·27
de Kruis ..	8·59	13·69

defined than they are to-day. Even in the lower reaches of drainage systems with a catchment area of several thousands of square miles river channels were either small or entirely non-existent. Large portions of the valley bottoms were almost level in cross section, had small gradients longitudinally, and the drainage water spread out in a thin film over the surface. Under these conditions the valley bottoms were covered with grass and shrubs, and the lowest points in the valley were marked by a succession of pools, known to the old Dutch settlers as "Zeekoe Gats," or "Hippopotamus Pools." These were fringed with a heavy growth of high reeds. Along steeper sections of the river the channels had but a fraction of their present cross section and were bordered with heavy growths of dense bush and forest. In course of time burning and the

concentrated grazing and tramping of sheep, goats, and cattle caused progressive desiccation. To-day the broad valleys are drained by deep and wide river channels, and the rich grass flats are dried out and eroded. Rivers only a few feet wide a generation ago are now two or three hundred feet in width. The dense bush in the valleys has disappeared, and the river channels, now ten or twenty times the size they used to be, present an unobstructed passage to flood water entering them.

Under these circumstances it is obvious that whilst the percentage of rain water which runs off is now far greater than in former times the drainage facilities have been enormously increased. Rivers, formerly perennial, now only run during a very small period of the year, and seldom for more than a few days at a time.

Given sufficient rain to produce saturation, a catchment covered with dense vegetation will yield eventually as high a rate of run-off as a bare catchment, and exceptional rains in the olden days must, with the restricted drainage system then existing, have produced much more serious floods than similar rains would produce to-day.

These facts go far to explain old flood marks handed down by tradition, which are clearly impossible if their discharges are calculated on the basis of the river channels now existing.

Such being the case, flood records both in terms of height and of discharge are reliable only if observed during comparatively recent years, or so long as the river channels are known to have been more or less stable. In Europe, where conditions have been stationary for centuries, records of ancient floods are often of great value, but in the new countries, especially if they are warm and dry, records based upon tradition or upon the memory of the " oldest inhabitant " are in many cases not only valueless but grossly misleading.

To show how difficult it is to predict maximum discharges a case is quoted where a board of experts, after carefully studying the history of the Sacramento River extending over a period of forty years, decided that the greatest flood to be expected was 80,000 cusecs. A few years later a flood exceeding 200,000 cusecs passed the same place.

Time after time carefully made estimates of maximum discharges have been falsified by floods observed subsequently. In South Africa many works constructed during the past twenty

years have been sorely taxed, and in a few cases have escaped destruction by very narrow margins. Old flood marks, tradition, and records of over a hundred years having proved to be wholly unreliable, what is the engineer to do ? He is, moreover, faced with this difficulty in a new and partly developed country : he must decide exactly in how far he is justified in taking risks. He must carefully consider the degree of damage likely to be caused by the destruction of his works by an exceptional flood. If, for instance, the effect of a washaway of a railway, road, or small dam would cause comparatively small damage below his works, and was not likely to involve loss of life, the amount of capital which he would be justified in expending in insurance is strictly limited. Such capital expenditure is often greater than the cost of rebuilding or repairing the work several times over if washed away or badly damaged. In other cases a very heavy expenditure in insurance is obligatory.

Many attempts have been made by engineers to produce a formula for the rate of maximum run-off, and the earliest and best known are due to irrigation engineers in India. The factors which determine the volume and intensity of discharge from a catchment are so numerous and so diverse that it is quite impossible to deduce any formula for the maximum intensity of discharge which will be applicable except within very narrow limits. The most that can be hoped for is that such formulæ will give for specific meteorological, topographical, and physical conditions an approximate value of the maximum intensity of high flood discharge, which will tend to minimize errors in the design of works for dealing with high floods.

In a paper published in the " Minutes of Proceedings " of the Institution of Civil Engineers in 1891, Vol. CXXXIV, Mr. Chamier laid down the following principle with regard to maximum run-offs :—

The maximum flood will generally be produced by the greatest possible rainfall of a duration corresponding to the length of time taken for the water to reach the point of discharge from the most distant point of the watershed. This general principle may be accepted as correct ; and as pointed out by G. B. Williams, M. Inst. C.E., in " The Engineer " of September 29, 1922, in an article on " Flood Discharge and Dimensions of Spillways in India," it follows that in order to discover the maximum possible flood from any catchment area it is necessary to know three things : (1) The period of concentration of flow from

the land within the watershed; (2) the maximum possible rainfall during that period; (3) the percentage of that rainfall to be allowed for.

These three factors should be capable of estimation. Williams in the article quoted above, gives an expression for the period of concentration of the flood, namely, time of concentration in hours, as a function of the area of the catchment in square miles, and the average gradient of the catchment from watershed to discharge point, but his formula is applicable to approximately circular catchments only.

He likewise gives a table of the maximum possible intensity of rainfall in inches per hour for eastern India, which is of interest as an example, though not applicable elsewhere.

TABLE XVII

Duration of Storm	Maximum Rate in Inches per Hour		
	Mean Rainfall 30 to 60 ins. per Annum	Mean Rainfall 60 to 100 ins. per Annum	Mean Rainfall over 100 ins. per Annum
5 minutes	6	9	12
15 ,,	$4\frac{1}{2}$	7	10
30 ,,	$3\frac{3}{4}$	$5\frac{1}{2}$	8
45 ,,	3	$4\frac{1}{2}$	6
1 hour	$2\frac{1}{4}$	$3\frac{1}{2}$	5
$1\frac{1}{2}$ hours	$1\frac{1}{2}$	$2\frac{1}{2}$	$3\frac{1}{2}$
2 ,,	$1\frac{1}{8}$	2	$2\frac{3}{4}$
3 ,,	1	$1\frac{3}{4}$	$2\frac{1}{2}$
6 ,,	$\frac{3}{4}$	$1\frac{1}{2}$	2
12 ,,	$\frac{1}{2}$	$\frac{3}{4}$	$1\frac{1}{4}$

It should not be impossible to construct probability curves on similar lines for other areas.

The rates of run-off for different catchments present the most uncertain factors, but reasonably correct estimates should in many cases be possible.

None of the formulæ in common use are rational, and most of them express the maximum run-off as a function of the area of the catchment only, and ignore all other contributing factors,

reliance being placed in every case on the adoption of a suitable coefficient.

In the present state of our knowledge of the subject, and in view of the large number of factors which must of necessity remain indefinite, it is doubtful whether it is worth while attempting to evolve a rational formula for maximum run-off.

Most of the formulæ are of the type

$$Q = C \times (\text{area})^n,$$

where Q = total maximum discharge in cusecs.

C=a coefficient representing the combined effect of the intensity of rainfall and the character of the surface of the catchment, and the power n, which is less than unity, appears to depend more upon the slope of the stream and its catchment area than on the other factors.

Formulæ of this kind are very numerous, such as

Col. Dickens: $Q = CM^{3/4}$

Ryves: $Q = CM^{2/3}$

Craig's formula takes into account the varying width of the catchments, their slopes, but not the intensity, which is an important factor. The formula is more rational than the preceding one. It is:

$$Q = 440B \times N \times \text{hyp. log.} \frac{8L^2}{B},$$

where B = mean width of the drainage area in miles;
L = length of catchment in miles;
$N = C \times v \times i$, where C is a coefficient;
v = velocity of the drainage in feet per second; and
i = rainfall in inches.

Rhind's formula is:

$$Q = 1300 \frac{M}{L^{2/3}},$$

the notation being as above. The above formulæ are given as examples, and are all of Indian origin, but there are many others.

These are fully treated in modern books on irrigation, which also give particulars of the coefficients which appear to suit individual catchments.

The value of any formula depends entirely upon the assumption of a correct coefficient, and this is just where they break down, as all such formulæ are founded on the basis of discharges actually observed or calculated, which, as has been shown, are but a poor criterion of what may possibly be expected.

The use of none of the above-named formulæ is recommended in South Africa, or in countries presenting similar conditions.

One of the latest flood formulæ has been suggested by J. T. Whistler as the result of an examination of some 600 records of American and European rivers and streams. The formula is :

$$Q \text{ max.} = c\left(\frac{10000}{M + 100}\right) + 5 \text{ cusecs per square mile,}$$

and

$$Q = c\left(\frac{10000}{M + 1000}\right) + 5 \text{ cusecs per square mile.}$$

The first formula represents the absolute maximum flood discharge during the worst twenty-four hours. The value of C varies from approximately 1·0 to 5·0. The expressions were derived from actual curves plotted on the basis of data available. These are nearly all in respect of catchments not exceeding 5,000 square miles, and are mostly for rivers and streams in the western States of America. European and eastern United States data were introduced because of their longer records, therefore indicating the possibilities for the streams of the western United States.

Conditions in the western States and in South Africa are to a large extent similar, and Whistler's researches form, therefore, a useful basis to work on.

It is now customary to record for the daily flood discharge the mean for the twenty-four hours' discharge. Whistler points out that the difference between this and the actual maximum or peak discharge which may often be for one hour or less, is small for streams with a catchment area in excess of 5,000 square miles and is negligible for much larger catchments. In the case of smaller catchments the peak discharge may be much in excess of the maximum twenty-four hour discharge, but these discharges are generally of short duration. When we are concerned only with the calculation of maximum spillway capacities the balancing effect of the reservoir storage between the spillway crest level and the maximum permissible high flood level, which is often very great, must be taken into account, and discretion must be used as to whether the formula should be used for the absolute maximum flood discharge or for the mean of the worst twenty-four hours.

This will depend upon the size of the catchment area and of the amount of reserve storage capacity.

The last quantity in the formulæ has been used as the maximum for all western United States streams by Mr. Whistler. It should, strictly speaking, vary somewhat as the coefficient C. According to the curves on which these formulæ are based C is 5·5 for "Great Basin" streams and 18 for Pacific coast streams, in the case of both the twenty-four-hour mean discharge and the absolute maximum discharge.

A beginning was made recently in the Union Irrigation Department to apply Whistler's curve for peak discharge to South African rivers, the curves being drawn on the basis of recent and more or less reliable flood discharges. It is clear, however, that we are at present short of data, especially in the drier parts of the Union, viz., in the Cape and the Orange Free State provinces. Further investigation during the next few years should produce valuable results. In Table XVIII are shown most of the maximum flood discharges which have been reliably gauged in South Africa. For the preparation of Whistler curves it is necessary to obtain data relating to smaller catchment areas.

It must be borne in mind that the accurate gauging of very high floods is a matter of considerable difficulty, especially when the flood rises to such a height as to overflow its banks and inundate large areas of adjoining land. So far as investigations have been carried $C=4$ or 5 are values which represent mean conditions which could be used in many areas. Only in case of flat, sandy catchments with low rainfall intensity, or on dolomite catchments, would $C=1$ be adopted. On the other hand, no records of floods in South African rivers exist at present justifying the assumption of a greater value for C than 14.

Whilst the development and use of Whistler's formula in South Africa appears justifiable, it is advisable to allow for the present a considerable margin of safety above the calculated maximum flood discharge, and it is further advisable to work with the formula for an absolute maximum discharge instead of using that for the greatest mean twenty-four-hour discharge.

Having determined the maximum flood run-off there remains the important problem of its disposal when designing storage dams, barrages or similar structures.

Large reservoirs generally have a very large storage capacity between full supply level and high flood level. In many cases the contents between these levels is equal to or even exceeds the entire capacity below full supply level. This reserve flood

TABLE XVIII

List of Maximum Flood Discharges Reliably Gauged.

Type of Catchment	Name of River	District	Area of Catchment. Sq. mile	Maximum Discharge Gauged	Cusecs per Sq. mile
Karroo Catchments Cape and Southern O.F.S.	Vlei spruit	Sterkstroom	12	12,900	1,075·0
	Bloem spruit	Bloemfontein	14	15,750	1,125·0
	Groot River	Prince Albert	168	16,450	98·0
	Kraai	Aberdeen	228	32,000	140·3
	Stormberg	Albert	970	55,800	57·5
	Traka	Uniondale	1159	40,000	34·5
	Sundays	Graaff Reinet	1,477	31,000	21·0
	Tarka	Tarkastad and Cradock	1,885	11,800	6·3
	Gamka	Prince Albert	4,055	30,200	7·4
	Riet	O.F.S.	4,542	22,200	4·9
	Sundays	Lake Mentz	6,150	131,000	21·3
Transvaal and Northern O.F.S. High Veld	Magalies	Krugersdorp, Pretoria	424	54,400	128·3
	Bronkhorstspruit	Pretoria	612	24,745	40·4
	Crocodile	Johannesburg, Pretoria	797	96,430	121·0
	Crocodile and Hennops	Do. Do.	1,117	108,000	96·6
	Great Olifants	Middelburg, Transvaal	1,502	46,850	31·2
	Klip	Vrede, O.F.S.	1,616	30,850	19·7
	Valsch	Kroonstad	2,230	77,000	34·5
	Modder	Bloemfontein	2,600	113,000	43·5
	Vaal (Standerton)	Standerton	3,923	80,728	20·6
	Wilge	Frankfort	5,816	100,000	17·2
	Vaal (Vereeniging)	Transvaal	16,500	118,600	7·2
	Orange (Aliwal N.)	Basutoland, Cape	14,000	220,000	15·7
	Orange (Orange R. Stn.)	and So. O.F.S.	36,500	281,000	7·7
	Rhenoster	Heilbron, Kopjes	1,050	70,000	67·0
Transvaal. Middle Veld	White	Barberton	42	4,572	109·0
	Plat	Waterbury	50	4,765	95·3
	Hex	Rustenberg	198	24,980	126·2
	Pienaars	Pretoria	417	43,000	103·0
	Komati	Carolina, Barberton, Ermelo	2,092	120,600	57·6
Mountainous	Blijde	Pearston, Cape	53	45,000	849·0
	Blijde	Lydenburg, Transvaal	200	16,850	84·3
	Nels	Oudtshoorn	72	3,500	48·6
	Kamanassie	Oudtshoorn	606	97,280	160·5
Dolomite	Klein Marico	Zeerust	594	17,950	30·2
	Mooi	Potchefstroom	816	4,410	5·4
	Schoonspruit	Klerksdorp	1,369	9,590	7·0

storage capacity produces a considerable equalizing effect, and in consequence it is not necessary that the waste weir or sluices for passing surplus water should be required to pass off a flood as rapidly as it arrives at the reservoir. Works for the disposal of large volumes of flood water are frequently very difficult to design, and are, in the majority of cases, very costly. If by utilizing the balancing effect of the reservoir the waste weir or discharge sluices can be appreciably reduced in size the cost of the project may be materially reduced. It is, therefore, of great importance to investigate this question closely.

The problem is by no means easy, and in all cases involves making certain assumptions regarding the rate of influx of the flood into the reservoir. With such assumptions the problem is capable of mathematical analysis.

By far the fullest and most rational treatment has been given to the subject in India, and was admirably dealt with some years ago by Captain A. ff. Garrett, R.E., in a Government of India Technical publication entitled "The General Theory of the Storage Capacity and Flood Regulation of Reservoirs." The paper is lengthy and contains advanced mathematics, and the derivation of the final working formulæ from the general formulæ will not be given here. For these the reader is referred to the original paper. The discussion of this important subject which follows, and also the practical examples given, are taken direct from the official publication above referred to.

GENERAL EQUATION

Let y = height of water in feet above any assumed datum.
A_y = area of contour in square feet at y.
C = capacity of reservoir in cubic feet from zero up to y.
h = maximum depth of water over waste weir or difference between F.S.L. and H.F.L.
i = flood influx in cusecs (assumed to be constant).
D = total discharge in cusecs from reservoir over waste weir or through sluices.
l = length of waste weir in feet.
t = time in seconds (from some assumed epoch when y is zero) at which level of water is y.

As set forth in Chapter VI in the section on calculation of

reservoir areas and capacities, the general equation for the cubic contents of a reservoir in cubic feet from zero up to y is :

$$C_y = \int A_y \, dy \quad \ldots\ldots (1)$$

$$C_y = ay + \tfrac{1}{2} by^2 + \tfrac{1}{3} cy^3 + \tfrac{1}{4} dy^4 + \ldots + k$$

Let the water level be y at any time t. Then if i be the influx into the reservoir, i.e. the rate at which the water is filling the reservoir we have

$$\frac{dC}{dt} = i ;$$

and if D be the total discharge from the reservoir (over the waste weir or through sluices) the net influx into it will be $i-D$, and

$$\frac{dC}{dt} = i - D;$$

or

$$\frac{dC}{dy} \times \frac{dy}{dt} = i - D. \quad \ldots\ldots (2)$$

From equation (1) $\dfrac{dC}{dy} = A_y$.

Therefore, substituting and transposing,

$$\frac{dy}{dt} = \frac{i - D}{A_y} \quad \ldots\ldots (3)$$

and

$$t = \int \frac{A_y \, dy}{i - D} \quad \ldots\ldots (4)$$

DISCHARGE OVER WASTE WEIRS WITH A FREE FALL

In these investigations take the crest of the waste weir as datum, and as the range of water level between full supply level F.S.L. and high flood level H.F.L. is comparatively small, we may put :

$$A_y = A_o + ky$$

without material error. Where A_o is the contour area at F.S.L.

if A_h = contour area at H.F.L.,

then $A_h = A_o + kh$.

Whence

$$k = \frac{1}{h}\left(A_h - A_o\right) \quad \ldots\ldots (5)$$

and k can be calculated for any particular reservoir. The discharge over a weir is $cly^{3/2}$, where c is a constant depending

on the form of the weir. Substituting their values the general equation (4) becomes:

$$t = \int \left(\frac{A_o + ky}{i - cly^{3/2}} \right) dy \qquad \ldots\ldots(6)$$

In the solution of equation (6), whereby the time in which the flood will rise to any given height h over the weir may be directly calculated, it will be assumed that c remains constant and independent of y. This is not strictly correct, but it is near enough for all practical purposes. The integration of the right hand number of equation (6) gives the following formula :—

$$T = \frac{2h}{i}\left[A_o B + khV \right] \qquad \ldots\ldots(7)$$

where $$B = \frac{1}{Z^2}\left\{ \frac{1}{6} \log \frac{1 + Z + Z^2}{(1 - Z)^2} - \frac{1}{\sqrt{3}} \tan^{-1} \frac{2Z + 1}{\sqrt{3}} + 0{\cdot}3023 \right\}$$

$$V = \frac{1}{Z^4}\left\{ \frac{1}{6} \log \frac{1 + Z + Z^2}{(1 - Z)^2} + \frac{1}{\sqrt{3}} \tan^{-1} \frac{2Z + 1}{\sqrt{3}} - 0{\cdot}3023 \right\}$$

and $$Z^3 = \frac{clh^{3/2}}{i} \qquad \ldots\ldots(8)$$

B and V, being functions of Z, only can be calculated and tabulated (*vide* table XIX) and the formulæ (7) and (8) will become simple to apply practically. The disadvantage of these formulæ is that a suitable value of T can only be found by trial and error after assuming provisional values for h and l.

An approximate formula, whereby l may be directly computed for any given value of h can, however, be established.

In equation (7) the term A_oB gives the time T, assuming the area of the reservoir to be constant from weir level to H.F.L. while the term khV gives the increment of time due to the increments in area with the rise in the water level.

If now we assume that the contour area between F.S.L. and H.F.L. to remain constant and equal to its mean value A_m, k will become zero and we have approximately:

$$T_1 = \frac{2h}{i} A_m B \qquad \ldots\ldots(9)$$

or $$B = \frac{iT_1}{2A_m h} \qquad \ldots\ldots(10)$$

If the conditions are that the flood does not rise above a certain height h in time T_1, B can be at once calculated. From the tables of B the corresponding values of Z^3 can be

found and the length l of the weir can be calculated from equation (8).

An investigation of the approximate formulæ No. 9 and No. 10 shows that the resulting value of l will be very nearly correct and will err slightly on the side of safety. Equation (8) may be written:

$$l = \frac{i}{ch^{3/2}} Z^3,$$

from which it is apparent that Z^3 is the coefficient of reduction in the length of the weir due to flood storage, for $\frac{i}{ch^{3/2}}$ is the usual expression for the length of a weir when the absorption effect of the reservoir is neglected. Hence, finally:

$$B = \frac{T \times i}{2h A_m} \quad \text{.....(12)}$$

$$l = \frac{n \times i}{ch^{3/2}} \quad \text{.....(13)}$$

when n is written for Z^3.

When the spillway consists of a drowned weir the discharge no longer varies as $h^{3/2}$, as is the case with a weir with a free overfall. The discharge varies more rapidly, but an approximation close enough for all practical purposes may be obtained by substituting for equation (13):

$$l = \frac{n \times i}{\triangle} \quad \text{.....(14)}$$

where $\triangle$ is the discharge per foot run of the particular weir in question with h feet passing over the crest, or, expressed in another form, if L is the length of the weir required to discharge i cusecs, then $l = nL$ is the length required to discharge the maximum flood of $n \times i$ cusecs, which has to be provided for if flood absorption is allowed for.

In the foregoing equations the passage of a single flood is only allowed for. It is possible, however, that two floods may succeed one another in rapid succession so that at the arrival of the second flood the water level is standing considerably above F.S.L. It is necessary to investigate, therefore, the time in which the water level will fall from H.F.L. to some lower safe level after the cessation of the first flood.

Replacing l by l^1 in the general equation (6), and putting

$i = o$, we obtain for the time of water level falling from h, or H.F.L., to a safe height above the weir crest h^1.

$$T^1 = \int_h^{h^1} \frac{A_o + ky}{-cl^1 y^{3/2}}\, dy$$

or
$$T^1 = \frac{2A_o + k\sqrt{hh^1}}{cl^1} \times \frac{\sqrt{h-h^1}}{\sqrt{hh^1}} \quad \ldots\ldots\ldots\ldots (15)$$

or if, as before, we replace $(A_o + k\sqrt{hh^1})$ by its approximate mean value A_m, then :

$$T^1 = \frac{2A_m}{cl^1} \times \frac{\sqrt{h} - \sqrt{h^1}}{\sqrt{hh^1}} \quad \ldots\ldots\ldots\ldots (16)$$

If $h^1 = o$, T^1 becomes infinite, or, in other words, it will theoretically take an infinite time for the water level to subside from H.F.L. to F.S.L. even after complete cessation of the flood influx. It is therefore necessary to consider how long the flood will take to subside to a " safe " level. This safe level must be arbitrarily assumed and will vary with the peculiar condition of each reservoir.

Capt. Garrett has assumed $0{\cdot}25h$ as a safe level, and for the purpose of this analysis we may accept his assumption. Then $h^1 = 0{\cdot}25h$. Then equation (16) takes the simple form :

$$T^1 = \frac{2A_m}{cl^1\sqrt{h}} = \frac{2A_m h}{l^1 \triangle} \quad \ldots\ldots\ldots\ldots (17)$$

It will be seen that the expression $\frac{\sqrt{h} - \sqrt{h^1}}{\sqrt{hh^1}}$ in equation (16) is equal to $h^{1/2}$ multiplied by a coefficient depending on the ration of h^1 to h, and if this coefficient is denoted by C then :

$$T^1 = \frac{2C A_m}{cl^1\sqrt{h}} = \frac{2C A_m}{l^1 \triangle} \quad \ldots\ldots\ldots\ldots (18)$$

See Table XIX (c) for the values of C.

Adhering to the assumption that $h^1 = 0{\cdot}25h$, we have from equation (17) :

$$l^1 = \frac{2A_m}{CT^1\sqrt{h}} = \frac{2A_m}{T^1 \triangle} \quad \ldots\ldots\ldots\ldots (19)$$

where $\triangle$ is the discharge per foot run of the weir when h feet of water are passing over the crest.

We have these two formulæ for the length of the weir, one based on the consideration that the flood must not rise above h over the crest, within time T, and the other that the flood

level must fall from H.F.L. to 0·25h above the crest in time T^1. Then from equations (13) and (19) we have :

$$\frac{l}{l^1} = \frac{n \times i \times T^1}{2h A_m},$$

and from equation (12) :

$$2h A_m = \frac{T \times i}{B};$$

therefore

$$\frac{l}{l^1} = \frac{T^1}{T} nB \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

Mr. Gurtu, Irrigation Engineer to the State of Gwalior, in a paper on "Flood Absorption in Tanks," published in 1909, first suggested that theoretically the best dimensions for a weir will be obtained by equalling l to l^1. With this condition equation so becomes :

$$\frac{T}{T^1} = nB \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

If B is known numerically n is also known and so is the ratio T/T^1. Hence knowing T and B, T^1 can be found from equation (21). For example, if T be taken as 24 hours, and for any given reservoir B is found to be 0·86, then from Table XIX(A) $n = 0{\cdot}78$ and $T/T = 0{\cdot}86 \times 0{\cdot}78 = 0{\cdot}667$, whence $T^1 = 36$ hours. Similarly for any other value of B, if T is taken as 24 hours, there is a corresponding value of T^1, and these values are shown in Table XIX (A). Hence we can see at a glance how many hours the weir will take to discharge from H.F.L. down to 0·25 h over the weir.

Assuming as a criterion of safety that the weir must discharge from H.F.L. to 0·25 h in 36 hours, B (calculated for a 24-hour flood) must equal 0·86.

Substituting in equation (12) we get as the best theoretical flood lift :

$$h_t \frac{T \times i}{2B \times A_m} = \frac{50000i}{A_m} \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

In the practical application of these formulæ it is advisable to provide for two contingencies viz., (1) for prolonged floods, (2) short periods of floods of very high intensity. In India a useful rule was laid down long ago, viz., that provision should be made for extended floods of twenty-four hours' duration, assuming a steady run-off of half an inch per hour from the entire catchment, and for short floods of high intensity lasting three hours. These latter were calculated by Dickens's formula.

These two conditions have been found suited to conditions in Central India, and would probably suffice for most South African conditions so far as time is concerned. But the maximum high flood discharge should not be calculated by Dickens's formula, but in accordance with the most suitable formula applicable to the district, or on more reliable data if available. In addition to the above conditions provision must be made for reducing the water level from H.F.L. to a safe level in a reasonable time, say twenty-four or thirty-six hours.

A practical problem would be to design a waste weir for a reservoir to satisfy the following conditions :—

(*a*) The weir must be able to accommodate for three hours T a flood of intensity I calculated by Whistler's formula :

$$I = c\left(\frac{10000}{M + 100}\right) + 5 \text{ cusecs per square mile of catchment.}$$

(*b*) It must also be able to accommodate a prolonged flood lasting twenty-four hours t of intensity $i = c\left(\frac{10000}{M + 1000}\right) + 5$ cusecs per square mile of catchment.

(*c*) After the cessation of the flood, the water level must fall from 8 feet h over the crest to $0{\cdot}25\ h = 2$ feet within 36 hours, T^1.

Assume $I = 6{,}664$ cusecs.
$i = 2{,}582$,,
$A_m = 38{\cdot}4$ million square feet.

First find the theoretical flood lift h_t to just satisfy conditions (*b*) and (*c*).

From equation (22)

$$h_t = \frac{50000i}{A_m} = 3{\cdot}4 \text{ feet.}$$

Thus if h is made anything less than 3·4 feet, the weir will be safe as regards condition (*c*). Assume that having regard to the nature of the site and other practical considerations $h = 3$ feet is a convenient figure to adopt, then the length of the weir will be calculated by equation (12) and equation (13).

Calculating first for the prolonged flood i lasting 24 hours t we have from equation (12)

$$B = \frac{t \times i}{2h A_m} = \frac{24 \times 60 \times 60 \times 2582}{2 \times 3 \times 38400000} = 0{\cdot}968.$$

Turning to Table XIX (A) we find that for $B = 0{\cdot}968$ the corresponding value of Z^3 or $n = 0{\cdot}854$, and also that T^1, or the time in which the weir will discharge from 3 feet over the crest to 0·75 feet, is 29 hours.

From equation (13), assuming the weir to have a free overfall and sharp crest :

$$l = \frac{n \times i}{ch^{3/2}} = \frac{0{\cdot}854 \times 2582}{3{\cdot}25 \times 3^{3/2}} = 130{\cdot}6 \text{ feet.}$$

Now recalculate for a three-hour high flood of 6,664 cusecs :

$$B = \frac{T \times I}{2h A_m} = \frac{3 \times 3600 \times 6664}{2 \times 3 \times 38400000} = 0{\cdot}312.$$

Turning to Table XIX (A) it is seen that if B is less than 0·5, n is zero or negative. This shows that no weir at all is required to accommodate such a flood for three hours. In other words, the flood storage capacity of the reservoir is greater than the volume which would flow in due to an influx of 6,664 cusecs persisting for three hours. Hence a weir 130·6 feet long with a three foot flood lift will satisfy all these conditions (*a*), (*b*), and (*c*).

Were flood storage neglected and the weir designed to pass the maximum discharge of 6,664 cusecs for an indefinite time the length of crest would have to be 395 feet, or three times as long as when flood storage is taken into account. If the waste weir is designed as a drowned weir, say with crest flush with the ground and spill channel with a slope of say 1 in 750, then the discharge per foot run of weir, with three feet of water running over the crest, is found to be 12·5 cusecs. Hence from equation (14)

$$l = \frac{n \times i}{\Delta} = \frac{0{\cdot}854 \times 2582}{12{\cdot}5} = 176 \text{ feet.}$$

The maximum discharge to be dealt with in the spill channel in this example will be

$$0{\cdot}854 \times 2582 = 2205 \text{ cusecs.}$$

Should it be found that the short flood necessitates a longer weir than the prolonged flood then the longer weir must be adopted.

In cases where it is essential to restrict the length of the weir the flood lift may have to be increased materially beyond the theoretical value given by equation (22). In such cases the calculations become more complicated and the problems must be solved by trial and error.

For example, making the same numerical assumption as before but taking $h = 5$ feet.

Then for prolonged floods the required length of the weir is found to be only 18 feet. By calculating for condition (*c*), however, with equation (17), the length works out at 84 feet.

It is clear, however, that if the weir is longer than 18 feet the water will never rise to H.F.L. at all. A correct solution can only be obtained by trial. Thus try a weir 50 feet long with a flood lift of 5 feet and suppose that at the commencement of a prolonged flood the water stands 2 feet over the crest.

To find the time taken for the flood to rise from 2 feet to 5 feet over the crest:

$$\text{From 0 to 5 feet, } n = \frac{lch^{3/2}}{i} = \frac{50 \times 3{\cdot}25 \times 5^{3/2}}{2582} = 0{\cdot}7.$$

From 0 to 2 feet, $n = 0{\cdot}18$.

Whence from Table XIX (B) $B = 0{\cdot}775$ and $0{\cdot}54$ respectively.

Assuming A_m constant at 39,600,000, we have from equation (12)

$$T(0 \text{ to } 5) = \frac{2hA_m}{i} B = 32{\cdot}7 \text{ hours.}$$

$$T(0 \text{ to } 2) = 9{\cdot}1 \text{ hours.}$$

Hence the water level will rise from 2 feet to 5 feet in 23·6 hours.

The time taken for water level to fall from 5 feet to 2 feet is given by equation (18).

$$T^1 = \frac{2CA_m}{cl^1\sqrt{h}}.$$

Here $h/h^1 = 2/5 = 0{\cdot}4$, and from Table XIX (C) $C = 0{\cdot}582$. Hence $T^1 = 35{\cdot}8$ hours.

Therefore a weir 50 feet long with a 5 foot flood lift will be safe.

Checking for a high 3-hour flood when 2 feet is running over the weir, we have

For 0 to 5 feet, $n = 0{\cdot}275$.

For 0 to 2 feet, $n = 0{\cdot}127$.

Whence $B = 0{\cdot}567$ and $0{\cdot}528$ respectively.

Hence $T(0 - 5) = 9{\cdot}3$ hours.

$T(0 - 2) = 3{\cdot}5$,,

Whence $T(2 - 5) = 5{\cdot}8$,,

As this is nearly double the time the flood is assumed to last, the 50-foot weir will be safe under all conditions.

TABLE XIX (A)

For Garrett's Flood Storage Formula.

B	*n*	*B*	*n*	*B*	*n*
·50	·0	·80	·728	1·10	·908
·51	·049	·81	·738	1·11	·911
·52	·095	·82	·748	1·12	·914
·53	·138	·83	·757	1·13	·917
·54	·178	·84	·766	1·14	·920
·55	·216	·85	·775	1·15	·923
·56	·252	·86	·783	1·16	·925
·57	·286	·87	·791	1·17	·928
·58	·318	·88	·799	1·18	·930
·59	·348	·89	·806	1·19	·932
·60	·377	·90	·813	1·20	·934
·61	·404	·91	·820	1·21	·936
·62	·429	·92	·826	1·22	·938
·63	·454	·93	·833	1·23	·940
·64	·477	·94	·839	1·24	·942
·65	·499	·95	·844	1·25	·944
·66	·520	·96	·850	1·26	·946
·67	·540	·97	·855	1·27	·948
·68	·559	·98	·860	1·28	·950
·69	·577	·99	·865	1·29	·952
·70	·594	1·00	·870	1·30	·954
·71	·610	1·01	·875	1·35	·960
·72	·626	1·02	·879	1·40	·965
·73	·641	1·03	·883	1·45	·970
·74	·655	1·04	·887	1·50	·974
·75	·669	1·05	·891	1·55	·978
·76	·682	1·06	·894	1·60	·981
·77	·694	1·07	·898	1·80	·990
·78	·706	1·08	·901	2·00	·994
·79	·717	1·09	·905	Infinite	1·000

TABLE XIX (B)

Garrett's Flood Discharge Formula. Table for B and V.

Z or *n*	*B*	*V*	*Z* or *n*	*B*	*V*
·00	·5000	·2500	·50	·6503	·3628
·01	·5020	·2514	·51	·6550	·3665
·02	·5040	·2528	·52	·6598	·3703
·03	·5061	·2543	·53	·6648	·3743
·04	·5082	·2558	·54	·6699	·3783
·05	·5103	·2573	·55	·6751	·3824
·06	·5124	·2588	·56	·6805	·3867
·07	·5146	·2604	·57	·6860	·3911
·08	·5168	·2620	·58	·6917	·3956
·09	·5190	·2636	·59	·6975	·4003
·10	·5213	·2652	·60	·7035	·4051
·11	·5236	·2669	·61	·7096	·4101
·12	·5260	·2686	·62	·7161	·4152
·13	·5283	·2703	·63	·7227	·4205
·14	·5307	·2722	·64	·7294	·4260
·15	·5331	·2739	·65	·7363	·4317
·16	·5356	·2757	·66	·7434	·4376
·17	·5381	·2775	·67	·7508	·4438
·18	·5406	·2794	·68	·7585	·4502
·19	·5432	·2813	·69	·7666	·4568
·20	·5458	·2833	·70	·7750	·4636
·21	·5485	·2853	·71	·7837	·4707
·22	·5512	·2873	·72	·7927	·4782
·23	·5539	·2893	·73	·8021	·4861
·24	·5567	·2914	·74	·8119	·4944
·25	·5595	·2935	·75	·8221	·5030
·26	·5624	·2956	·76	·8328	·5120
·27	·5653	·2978	·77	·8441	·5215
·28	·5683	·3000	·78	·8560	·5316
·29	·5713	·3022	·79	·8684	·5423

TABLE XIX (B) (*continued*)

Z or *n*	*B*	*V*	*Z* or *n*	*B*	*V*
·30	·5744	·3045	·80	·8815	·5536
·31	·5776	·3069	·81	·8954	·5656
·32	·5808	·3093	·82	·9101	·5783
·33	·5841	·3118	·83	·9257	·5919
·34	·5874	·3143	·84	·9425	·6065
·35	·5908	·3169	·85	·9604	·6223
·36	·5942	·3195	·86	·9797	·6394
·37	·5977	·3222	·87	1·0006	·6581
·38	·6012	·3249	·88	1·0234	·6785
·39	·6048	·3277	·89	1·0484	·7007
·40	·6085	·3305	·90	1·0760	·7255
·41	·6123	·3334	·91	1·1068	·7535
·42	·6162	·3364	·92	1·1415	·7851
·43	·6202	·3394	·93	1·1812	·8216
·44	·6242	·3425	·94	1·2276	·8646
·45	·6283	·3457	·95	1·2830	·9163
·46	·6325	·3489	·96	1·3487	·9810
·47	·6368	·3522	·97	1·4414	1·0659
·48	·6412	·3556	·98	1·5694	1·1894
·49	·6457	·3591	·99	1·7928	1·4064

TABLE XIX (C)

For Garrett's Flood Discharge Formula. Values of C.

Ratio h^1/h	C
0·0	Infinite
0·1	2·17
0·2	1·24
0·3	0·824
0·4	0·582
0·5	0·414
0·6	0·291
0·7	0·195
0·8	0·118
0·9	0·054
1·0	0·00

In an article by H. K. Palmer in the " Engineering News " (Vol. 67, No. 12) dealing with the same subject the same general equation for the time taken for the water level to rise over the spillway crest level is deduced, viz., equation No. (6). The author of the article states, however, that the integration of the equation is so complex that it is impracticable. He then proceeds to elaborate a graphical method of solving it. This method consists in drawing a curve in which values of y are plotted as abscissæ and values of

$$\left(\frac{A_o + ky}{i - cly^{3/2}}\right)$$

as ordinates. Thus the integration of this expression will be the area between the curve, the x-axis and the two ordinates representing the limits of integration. This area will give the time required for the water in the reservoir to rise from its initial height to the height of the upper limit of integration, viz., to H.F.L.

In order to work with a more suitable unit of time than seconds Palmer gives a modified formula based upon the assumption that 1 cusec = 2 acre feet per day, which is approximately correct. The expression could then be written

$$T = \int^{h} \tfrac{1}{2} \times \frac{A_o + ky}{i - cly^{3/2}} \times dy,$$

where T is expressed in days and A_o and k in acres.

In cases where reliable records exist which would give the form of the flood wave it is possible to apply the above method of solution to the flood curve, but the solution is complex and can only be done by a series of approximations.

It is first necessary to plot a curve showing the relation between the volume of water in the reservoir and the discharge over the spillway. The next step is to assume that at the end of a certain time the spillway will be discharging a certain amount. As the discharge is very small at first this interval can be made rather large, a quarter or half a day, for example. This point should be joined with the origin by an assumed discharge curve. Next measure the area between the two curves from the origin to the assumed point, thus getting the amount of water assumed to be in the reservoir. With this amount find from the discharge curve, plotted as explained above, what the actual discharge should be. If too great a discharge has been assumed in the first instance the volume of water in the reservoir will be

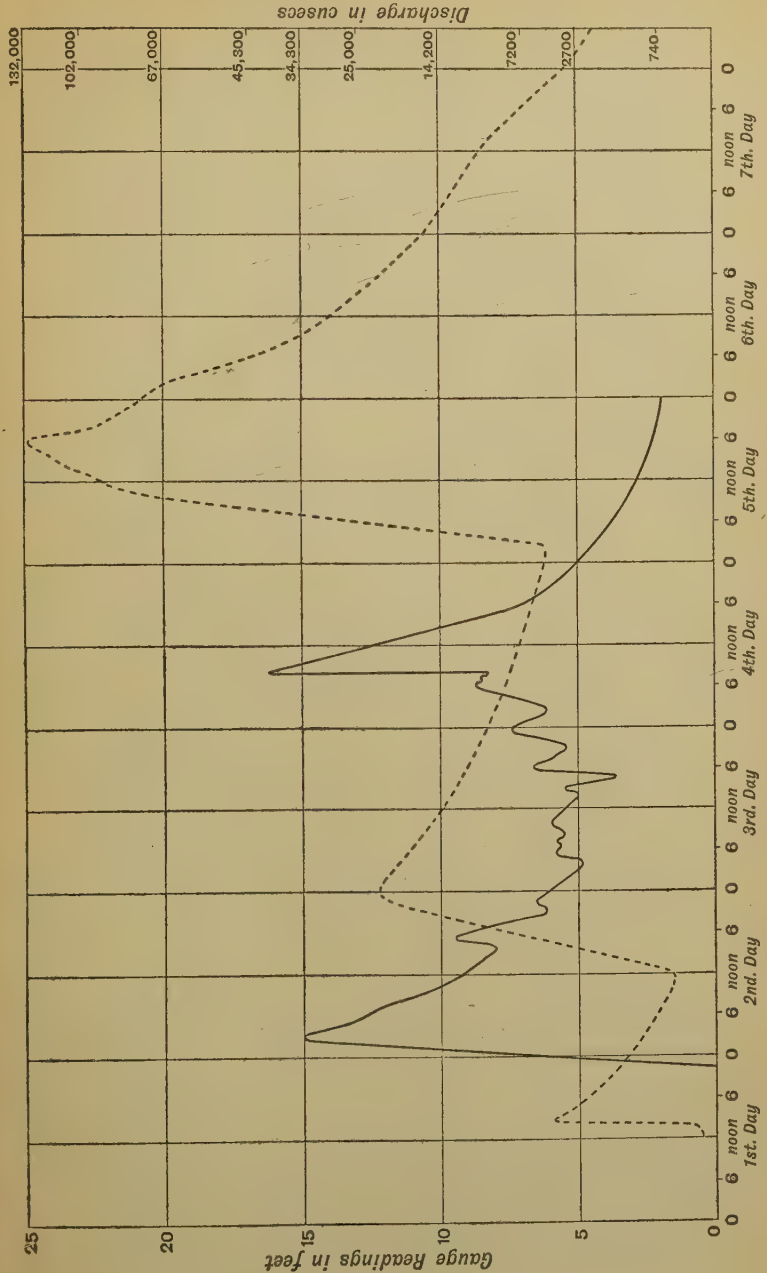


FIG. 9.—Records of two floods in Sundays River.

too small, and consequently the discharge over the spillway too small. Therefore the point should be located on the diagram somewhere between the assumed point and the one formed from the discharge curve. Draw the discharge curve to this point and find the actual discharge as before. Thus by trial and

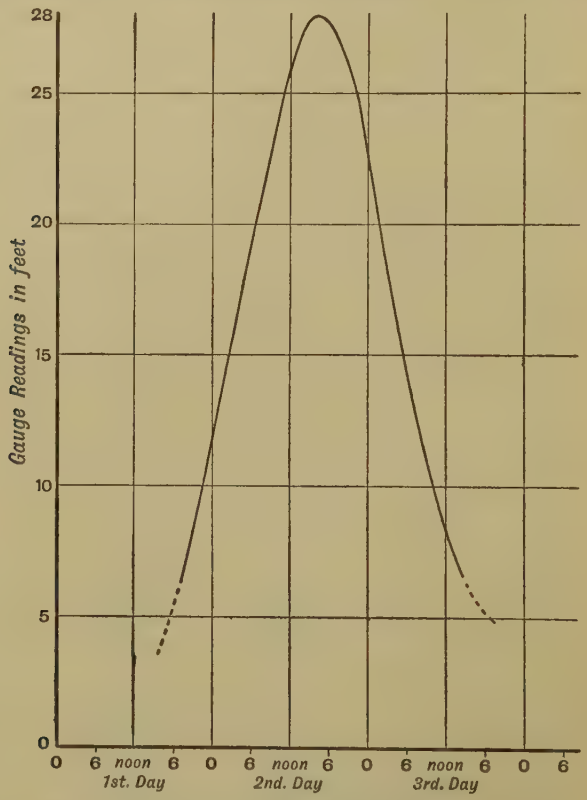


FIG. 10.—Record of flood in Modder River, 1920.

error continue the operation until a point is found which will give on the discharge curve the same value as was assumed in making the last trial. Having definitely located one point another should be located in the same manner, this time the assumed discharge curve being drawn from the point previously found to the new point; and the area between the two curves being measured always from the origin to the point in question. When the whole discharge curve is plotted as far as the maximum

discharge, it may be found that the assumed length of spillway does not give the desired proportions, in which case the whole process must be gone through again, using another length.

It will be seen that the method is very laborious, and it is only approximate when undertaken. In cases of this kind it is not possible to predict the form of flood curve for the worst conditions likely to occur. The peak discharge of the flood and

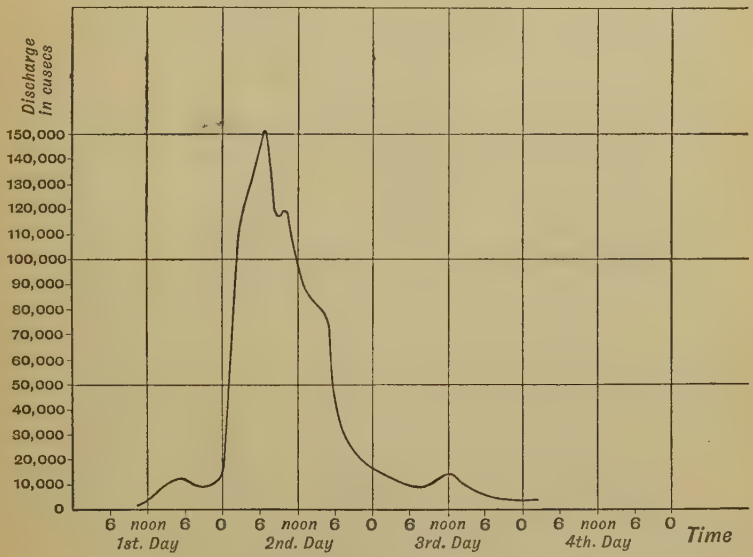


FIG. 11.—Record of flood in Crocodile River, 1909.

the length of its duration at that stage must be assumed in any case, and the assumption of the precise ratio of rise of the flood to the peak and fall after the peak has been passed is as arbitrary as it is unnecessary.

The solution of the problem by Garrett's method is comparatively simple, involves no unnecessary and complicated assumptions, and provides a more rapid solution than Palmer's graphical method.

By way of illustration of the behaviour of large floods graphs are annexed of two actual large floods (one being a record flood with a peak discharge of 131,000 cusecs) which were gauged during the years 1920 and about new year 1921-1922. The catchment area in this case is 6,150 square miles. A very

typical graph is also given for a very high flood in the Modder River near Bloemfontein from a catchment area of about 1,750 square miles. A graph of a different kind is also given representing the record flood in the Crocodile River near Pretoria in 1909 from a catchment area of 1,700 square miles. In this case the ordinates of the graph represent discharges in cusecs. All these graphs are compiled from charts made by self-recording gauges.

Information such as that shown in these graphs is of great importance when making assumptions of the rate of inflow with a reservoir for the purpose of calculating spillway capacities.

CHAPTER V

RESERVOIR AND CANAL LOSSES

When designing irrigation projects the estimate of losses of water, inevitable in any scheme comprising works for the storage or conservation of water, is a matter of very great importance, and the efficiency of such a scheme will depend very largely upon the correctness of this estimate.

Water losses are due to two main causes, viz., evaporation and absorption. From the point of view of practical working it is not possible to differentiate with any great degree of accuracy between the two, but in dealing with the subject evaporation will be defined generally as the loss which takes place from the surface of a large sheet of water, such as a reservoir, whereas absorption may be defined as the loss of water flowing in natural and artificial channels.

In the case of evaporation the loss takes place by water being absorbed at its surface by the atmosphere or by transpiration from the leaves of plants which are in actual contact with, or in close proximity to, the water, thus forming a medium of conveyance of a very considerable amount of water to the atmosphere.

Absorption, on the other hand, in its strict sense implies the loss of water into the subsoil by percolation. In the case of absorption much of the water, which in the first instance percolates into the soil, is subsequently returned to the surface of the ground adjoining the reservoir, river, or canal by capillary action and there evaporated, either at the ground surface or by transpiration through the leaves of the plants.

Exactly what proportion of the water is thus brought to the surface and evaporated, and what proportion sinks to the subsoil under the action of gravity, can sometimes be determined on a small scale by means of elaborate experiments, but on a big scale this proportion cannot be so determined. The forces

at work, both in the case of evaporation and absorption, are so numerous and variable that any mathematical treatment of the subject is impracticable, and the engineer must base his estimate of these losses largely upon empirical rules and experience.

In doing so it is of course necessary to have a clear understanding of the various forces involved and the manner in which these forces act, and these will be briefly set out.

All moisture falling from the atmosphere in the form of rain, snow, etc., has at some previous time been absorbed into the atmosphere by evaporation. All moisture thus evaporated from the surface of the globe will at some time or other again be precipitated in the form of rain, snow, etc. Taking the world as a whole, therefore, evaporation and precipitation balance one another.

Evaporation depends mainly upon the temperature, upon the relative humidity of the atmosphere, and upon wind. Evaporation from the surface varies enormously in different parts of the world, and at different times of the year. In regions of high relative humidity, whether in cold, foggy climates or in tropical steamy climates, evaporation is very low.

Conversely in hot, arid regions, such as parts of South Africa, Australia, Egypt, California, etc., evaporation is very high. In the two cases mentioned it may vary from as little as ten inches per annum, in cool and foggy regions such as Labrador, to over one hundred inches per annum in hot, arid countries.

The amount of evaporation is generally determined experimentally by means of small pans or tanks placed in such a manner that the water contained therein is, as nearly as possible, of the same temperature and exposure as that of the body of water of which the evaporation is to be measured. In such tanks, which must be entirely watertight, the amount of water taken into the atmosphere is due to surface evaporation only, all losses due to absorption, etc., being prevented by the impervious nature of the containing sides.

In nearly all cases where evaporation data are given they are the result of such pan or tank experiments. It has been found experimentally, however, that the results of such experiments do not represent truly the evaporation from large surfaces, such as reservoirs, lakes, etc. In 1909 to 1910 experiments were made by the United States Weather Bureau, in order to determine the relationship between tank evaporation and that from

large areas, the results of these experiments being briefly as follows:

From a pan, 2 feet in diameter, the evaporation was found to be 75 per cent greater than from a large area. From a 4-foot pan the evaporation was 50 per cent greater, and from a 6-foot pan 30 per cent greater.

It will be seen, therefore, that great caution must be exercised in applying the published results of evaporation experiments from pans or tanks to reservoirs and other large areas.

Another method of determining the evaporation is by means of instruments known as evaporometers. Such instruments are mainly used at observatories, meteorological stations, etc. To what extent data obtained from such instruments are accurate and applicable to large water surfaces, is not very clear.

In the year 1888 some experimental work was done by Mr. T. Russell, of the United States Signal Service, the instrument used being the "Piche" evaporometer. In commenting on these experiments Messrs. Davis and Wilson, in their book on Irrigation Engineering, state that while it is probable that results obtained with this instrument are not particularly accurate, the comparison of these results with those obtained by other methods in similar localities show such small discrepancies that they may be considered of value until superseded by results obtained by better methods. It may be assumed, therefore, that results obtained by means of efficient evaporometers are as reliable as results obtained from evaporation tanks.

The effect of wind on evaporation is very great. In the same experiment as quoted above the relative effect of different wind speeds on evaporation was determined, and it was discovered that with a wind velocity of five miles per hour the evaporation was 2·2 times that in quiet air, and increased speedily to the limit of the experiment, showing that with a wind velocity of thirty miles per hour the evaporation was 6·3 times that in quiet air.

It is generally assumed that the relative humidity has a very important bearing on the amount of evaporation, and in this connexion the researches of Mr. Desmond Fitzgerald, at one time Chief Engineer of the Waterworks at Boston, U.S.A., are of considerable interest and importance. Fitzgerald's researches are fully set forth in a paper contributed to the American

Society of Civil Engineers (Vol. XV, No. 340, page 581), and quoted also in "Proc. Inst. C.E.," Vol. LXXXVIII, page 516.

Inter alia Fitzgerald says that it is necessary not to fall into the error of supposing that the rate of evaporation for a pool of water bears any direct relation to the relative humidity of the air. This might be the case if the temperature of the surface of the water followed that of the air, even approximately, in its change of temperature. Many writers of the highest reputation have assumed this to be the case.

The rate of evaporation depends upon the force of vapour, and not upon the relative humidity. Continuous readings show the rate of evaporation practically constant during twenty-four hours, whereas the relative humidity, during the same period, varied from 68 per cent, between 8 and 9 p.m. to 34 per cent between noon and 1 p.m.

The difference between the temperature of the water and the dew point determines the evaporation. If the water is cooler than the dew point no evaporation takes place. On the contrary, the cooler water would condense some vapour in the air. If the dew point is lower than the water temperature evaporation must be taking place.

In Fitzgerald's opinion the order of importance is, first, termperature; second, wind. The influence of barometer changes is so small that they can be ignored. From his investigations Fitzgerald deduced a formula for the calculation of the amount of evaporation. In this expression

E = evaporation in inches per hour.

V = maximum force of vapour in inches of mercury corresponding to the temperature of the water.

v = force of vapour in the air.

w = velocity of wind in miles per hour.

$$E = [0{\cdot}014(V-v)-0{\cdot}0012(V-v)^2]\times(1+0{\cdot}67w^{3/5})$$

Curves plotted from the above formula are so nearly straight lines that these may be drawn through the plotted points of temperature and wind diagrams, instead of curves. This simplifies the formula, which becomes:

$$E = 0{\cdot}0166(V-v)(1+w/2)$$

$$E = \frac{(V-v)(1+w/2)}{60}$$

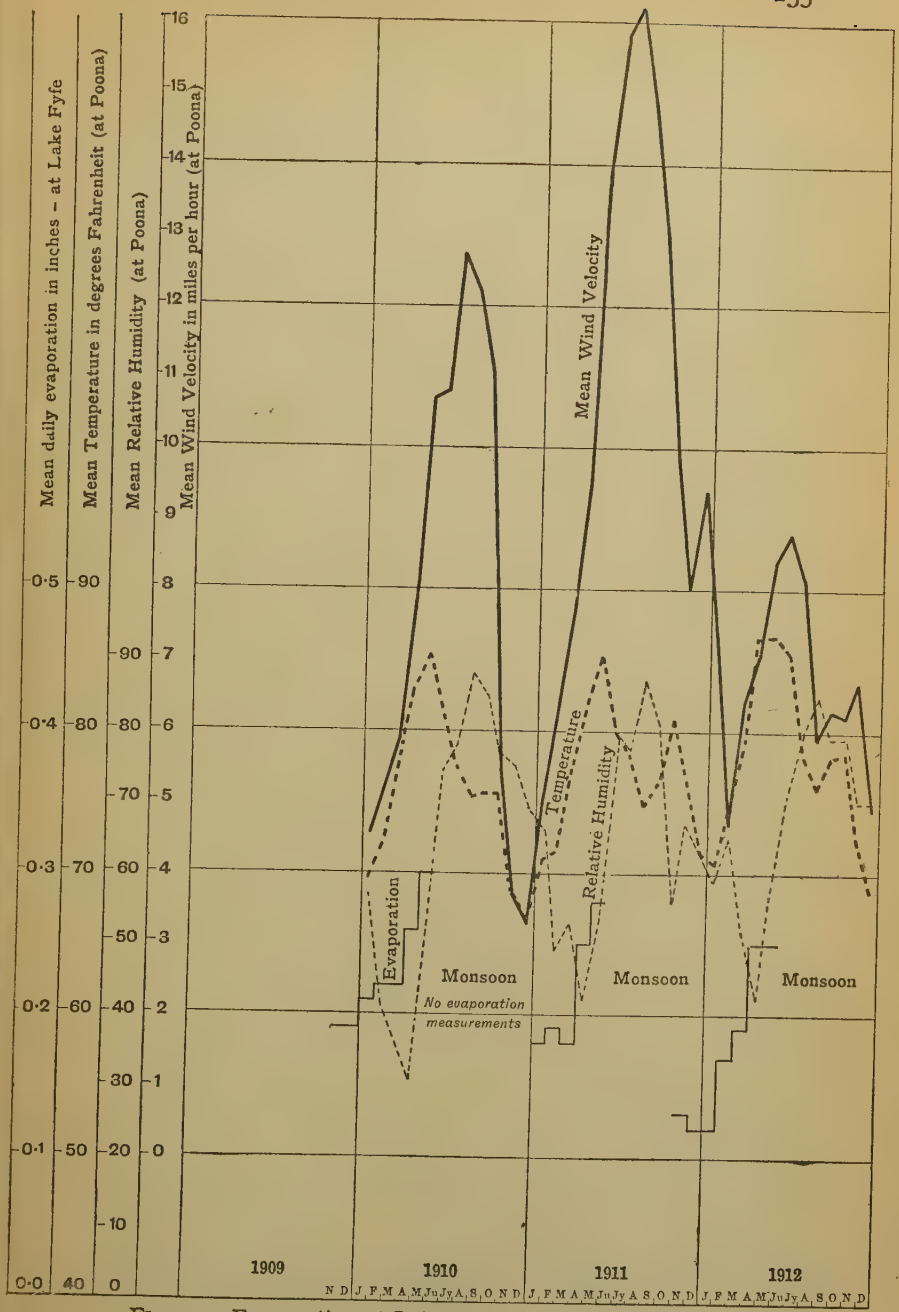

Mean daily evaporation in inches - at Lake Fyfe
Mean Temperature in degrees Fahrenheit (at Poona)
Mean Relative Humidity (at Poona)
Mean Wind Velocity in miles per hour (at Poona)
Mean Wind Velocity
Temperature
Relative Humidity
Evaporation
Monsoon
No evaporation measurements
Monsoon
Monsoon
1909
1910
1911
1912
N D J F M A M Ju Jy A S O N D J F M A M Ju Jy A S O N D J F M A M Ju Jy A S O N D

FIG. 12.—Evaporation at Lake Fyfe Reservoir.

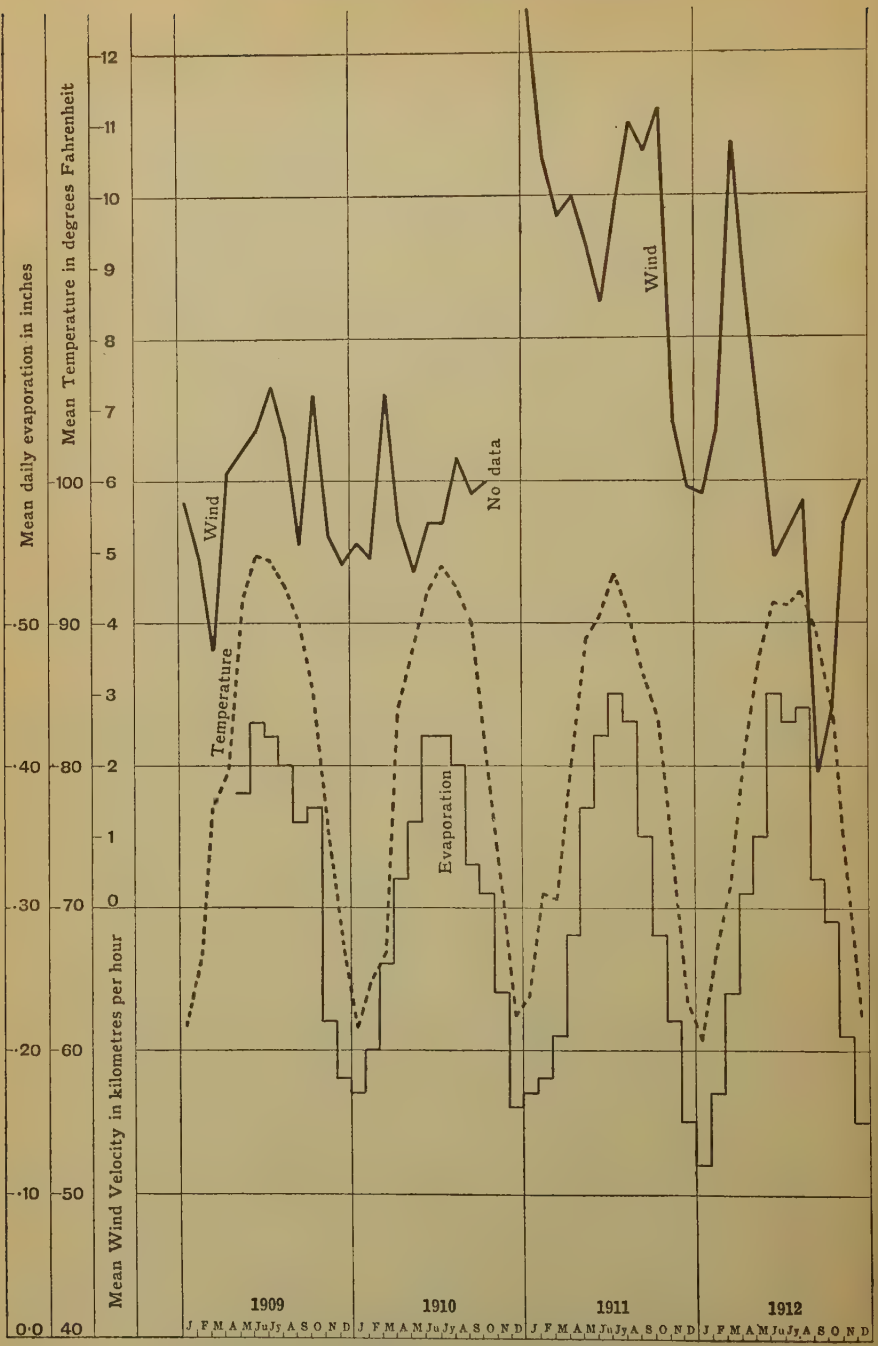


FIG. 13.—Evaporation at Assuan Reservoir.

This formula, compared with long-continued observations at Chestnut Hill Reservoir in Boston, agrees very closely.

It is unfortunately not possible to correlate properly the daily evaporation from a water surface with mean daily meteorological conditions. To obtain a rational relationship hourly comparisons are necessary.

J. R. Sutton, D.Sc., of Kimberley, who has devoted much study to this question, holds that the evaporation is a function of temperature only, and that other meteorological conditions are in the nature of perturbations. He points out also that nobody has as yet succeeded in determining the temperature of the immediate surface of still water, nor whether evaporation takes place from the actual surface or from the layer immediately below the surface.

The relationship between evaporation and temperature, wind, and relative humidity is clearly shown in the accompanying charts drawn for Assuan in Egypt, and for Lake Fyfe in Deccan, India (Figs. 12 and 13).

In the case of the Assuan reservoir the data are all taken from Buckley's " Irrigation Pocket Book." In the cases of the two large Indian reservoirs the data relating to temperature, wind velocity, and relative humidity are taken from the Indian meteorological reports, and are monthly means for Poona, the nearest reporting station. It is clear from the Assuan curves that the mean daily evaporation for each month can be expressed as a function of the mean monthly temperature within reasonable limits of accuracy. The independent effect of wind is by no means clear.

In the case of Lake Fyfe the relationship between mean evaporation and mean temperature is much the same as at Assuan, but in this case there is also an equally clear relationship between mean evaporation and mean wind velocity and mean relative humidity. In fact, the amount of mean daily evaporation varies directly on the mean temperature and mean wind velocity and inversely as the mean relative humidity.

No evaporation figures are given for Lake Fyfe for the five months from June to October, as during these monsoon months no evaporation data are available.

In the present state of our knowledge of the subject it is doubtful whether it is practicable to derive an expression for

the amount of evaporation in a small tank based upon mean temperature, wind velocity, etc., which is capable of application in other cases for which no data exist.

Evaporation figures obtained from tank measurements are not representative of the losses which take place in a large reservoir. In the latter case, as has been shown, the evaporation itself is less, but, on the other hand, there are considerable additional losses in a reservoir due to wave action on the margins which cause the water to spill in thin films over large areas of flat ground. From this wetted land the evaporation under the influences of heat and wind and low relative humidity is very great.

Absorption losses in a large area of water are difficult to determine, but in the case of properly constructed reservoirs, these are very small compared with evaporation losses, especially after a considerable amount of silt has been deposited on the floor thereof.

Taking all things into consideration, properly conducted tank experiments should serve as a fairly good guide to probable reservoir losses at the same site.

Evaporation varies very greatly in different places, according to the climatic conditions obtaining.

To give some idea of the amount of annual evaporation losses in countries somewhat similar to South Africa the annexed table has been compiled from various sources.

It will be seen that in hot and dry plateau areas somewhat similar to the Karroo in the Cape and Orange Free State Provinces the total annual losses range from 80 inches up to 164 inches per annum.

The figures quoted are, it is believed, nearly all derived from tank experiments, though the tank sizes are not given. In two cases the evaporation was measured with a " Piche " evaporometer.

Unfortunately there exist in South Africa very few data concerning evaporation in similar regions. The most useful discussion of the subject is contained in a paper read by Mr. A. D. Lewis (now Director of Irrigation for the Union of South Africa) on run-off and reservoir losses at Van Wijks Vlei (a hot and arid part of the Cape Province) before the South African Society of Civil Engineers (Vol. XIII, 1915).

With the exception of the years 1884 and 1885 it was only possible to determine the losses with reasonable accuracy

TABLE XX

Evaporation in Inches per Annum at Various Places in the Western United States, Egypt, India, and Australia.

Place	Inches per Annum	Remarks
Western United States of America (Inland)		
Salton Sea (1,500 feet inland)	164·5	Evaporation tank observation by U.S. Weather Bureau.
Do. (1,500 feet at sea)	108·6	
Indio (Cal.)	119·3	
Granite Reef (Arizona) ..	97·7	
Minedoka (Idaho)	96·5	
Elephant Butte (N.M.) ..	87·0	
United States (Coastal)		
Sweetwater (California) ..	59·07	Evaporation tanks floating in reservoirs.
Arrowhead do.	39·55	
Rochester (New York) ..	34·31	
Boston (Mass.)	39·71	
Egypt		
Assuan 1910	109·38	Evaporation from surface of reservoirs.
Do. 1911	107·00	
Do. 1912	106·08	
India		
Average at six reservoirs in the Deccan (Bombay) ..	96·34	
Australia, N.S.W.		
Murrumbidgee River Project.		
1914-15	45·0	Evaporation tanks and observations.
1915-16	39·18	
1916-17	31·48	
1917-18	29·00	
1918-19	40·33	

subsequent to 1909. Mr. Lewis gives the mean losses due to evaporation and normal absorption as follows:

	Jan.	Feb.	Mar.	Apl.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.
Ins.	$8\frac{3}{4}$	$7\frac{3}{4}$	$6\frac{1}{2}$	$4\frac{1}{2}$	$3\frac{3}{4}$	$2\frac{1}{2}$	$2\frac{1}{4}$	$3\frac{3}{4}$	$4\frac{3}{4}$	6	8	$8\frac{1}{2}$

Total = 67 inches

This figure is surprisingly low for such a locality. During the first two years in the history of this reservoir the losses are stated to have been at 88 and 76 inches respectively, but in both cases the absorption was probably abnormal.

During 1910 the conditions were, in Mr. Lewis's opinion, favourable for minimum loss, and in that year the total losses were at the rate of $62\frac{1}{2}$ inches per annum.

Other data for inland stations are as follows:

Vaal River, Kimberley Water Works (F. S. Lynch).

1910	1911	1912	1913	1914	1915	1916	1917	
91·7	86·4	90·3	89·6	90·0	91·6	92·9	78·9	Ins.

At Kimberley, Dr. Sutton reported an annual loss of only 55 inches from a tank 4 feet square and 30 inches deep immersed in a pond.

Tank experiments at Bloemfontein (G. Stewart, City Engineer) showed that in exposed situations the evaporation ranged from 66·46 to 82·32 inches per annum, and in sheltered positions between 47·93 to 57·69 inches per annum.

The mean evaporation at Johannesburg (Union Observatory) is 74·67 inches per annum.

Coming down from the plateaux into more coastal climatic conditions, the losses in the Pietermaritzburg and Port Elizabeth reservoirs vary between 32 and 46 inches per annum. The mean for Cape Town (Molteno Reservoir) is 78·57 per annum.

In drawing up storage projects in the Union of South Africa the following are reasonable allowances for evaporation and absorption losses: For coastal areas in Natal, Cape Province, South Coast, 48 inches. Over the rest of the Cape Province and over the whole of the Orange Free State, 60 to 80 inches. Central and eastern Transvaal, 50 to 60 inches. Western Transvaal, 50 to 70 inches. These are only average figures. In individual

cases considerable discretion must be exercised, and higher or lower figures adopted in special cases. The figures given are, however, believed to be reasonable allowances to make.

When calculating the annual evaporation and absorption losses in terms of volume the condition under which the reservoir will be worked must be taken into consideration.

The usual procedure is to calculate the losses by multiplying the depth of annual evaporation allowed by the mean area of the reservoir. This is correct only on the assumption that the reservoir is filled up to full supply level and drawn down to bottom level at the dam annually.

If the outlet sill is some height above the lowest point in the reservoir, as is generally the case, the mean should be calculated between outlet sill level and full supply level.

If the reservoir is made large enough to store more than one year's supply, or to balance supplies of good years against bad years, the particular contour areas on which the losses are based must be carefully chosen to represent average conditions of withdrawal and replenishment of the water impounded.

In most cases in South Africa conditions require that the reservoir should be maintained full during the winter months, when the evaporation is least, and drawn down most heavily during the driest and hottest months, when evaporation is greatest.

In countries such as South Africa it is not economical to store water for periods exceeding two years, and Strange in his book on Indian Storage Reservoirs maintains that in India storage for even two years is uneconomical. The losses from evaporation, etc., in the higher contours of the reservoir are very great and constitute a large proportion of the total available storage capacity.

ABSORPTION

All earth surfaces, whether natural or artificial, absorb water, if brought into contact with it. This absorption by the soil accounts mainly for the losses of water carried in earth canals. There are, of course, evaporation losses from the surface as well, but these form so small a percentage of the absorption losses that, for practical purposes, they may be neglected.

Absorption, or seepage, losses vary enormously according to the nature of the ground in which the canal is cut, or according

to the character of the material forming the banks. In the same class of soil, provided it is not of a sandy character, a canal in embankment will often lose less water from absorption than a canal in cutting in the same material, this being due to the fact that the density of a well-made bank is greater than that of the natural soil from which the earth is taken.

Speaking generally, seepage losses are greater in canals cut in fissured rock, gravel, or coarse sand than in loamy soils or clays. A clay soil may present a greater aggregate area of interstices between soil particles than a sandy soil, but resists better the passage of water through it owing to the extreme smallness of the pores.

A new canal invariably leaks badly, and wherever possible it should be encouraged to cover itself with a layer of fine silt, which will act as a natural waterproofing. In an earth channel in which no such silt deposit takes place its efficiency as an aqueduct decreases with age.

Other things being equal, the rate of seepage tends to increase with the increased temperature of the water.

Seepage losses have been the subject of much investigation by canal engineers in many countries, but the most valuable data, so far as South Africa is concerned, have been obtained from investigations made in Upper India and in the United States of America.

In view of the enormous aggregate mileage of irrigation channels in India it is natural that the subject of absorption losses in canals and distributaries should have received close attention over a period of many years.

So far as the great Punjab canal systems are concerned all past investigations were summarized in 1917 in a couple of official memoranda by F. W. Woods, the Chief Engineer, Irrigation Works, Punjab. These memoranda deal separately with canals having a capacity exceeding 250 cusecs, and reaching up to 11,600 cusecs, and with "minor" channels and watercourses respectively.

As a result of these investigations absorption losses are now calculated by the formula:

$$Q = c \times a \times d,$$

where Q = absorption loss in cusecs.

c = coefficient varying inversely with the hydraulic mean depth.

a = area in millions of square feet of the reduced

wetted surface of the channel, i.e. of the bed of the channel, plus one-half of its side slopes.

d = depth of water in feet.

The factor d in the formula represents the head of water pressure on the wetted surface of the channel, and this is the same thing as the depth of water on the bed of the channel. In special cases, where the spring level is above the bed level of the channel, d should be taken to be the difference in level between the canal water surface and the spring level just outside the channel.

Extensive tables are given in the memoranda showing the absorption losses in different reaches of the canals and branches throughout the Punjab, from which certain average figures relating to each of the different canal systems are here produced in Table XXI. The rate of absorption loss is calculated, in the first instance, per million square feet of reduced wetted surface (*vide* Col. 10 of the statement), but as the practice in the past has been to calculate the loss per unit of the entire wetted surface, i.e. including the whole of the side slopes, the rate of loss is expressed in terms of this unit in Col. 13.

Absorption losses are likewise expressed in terms of percentage of full supply capacity, calculated over the entire length of the canal system dealt with.

It will be seen from Col. 13 that the rate of total loss per million square feet of wetted surface varies, under full supply conditions, from 9 to 10 cusecs, a rate of loss which is considerably greater than was formerly assumed for canals in the Punjab, viz., 8 cusecs, a figure which will be found quoted in numerous books on irrigation. This figure of 8 cusecs per million square feet of wetted perimeter was formerly used also in the case of smaller canals.

Recent investigations have shown, however, that this assumption gives very erroneous and, generally, excessively high results in the case of minor distributaries, and it has been demonstrated that the new formula $Q = c \times a \times d$, quoted above, gives results which agree fairly well with data of actual experimental observations. For small canals, in good regime, the absorption losses would appear to be of the order of from 4 to 6 cusecs per million square feet of total wetted areas.

In American practice the tendency is to express seepage losses either as a " per mile " percentage, or as a loss in cubic feet per square foot per twenty-four hours. Davis and Wilson,

TABLE XXI

Abstract of Statistics of Design Governing Absorption Losses in the Principal Punjab Canals.

AVERAGES OF EACH CANAL SYSTEM					
Names of Canals	Reckoning from Mile No.	Full Capacity at Head (cusecs)	Length of Main and Branch Canals (canal miles)	Bed Width (feet)	Full Water Depth (feet)
1	2	3	4	5	6
Sirhind	0	8,000	481·8	59	7·4
do.	26	7,673	455·8	51	7·2
Upper Bari Doab ..	0	6,500	339·8	65	6·4
Lower Chenab	0	10,990	427·4	79	7·4
do.	8	10,730	419·4	76	7·4
Lower Jhelum	0	4,000	221·8	63	5·4
do.	14·2	3,900	207·6	—	—
Lower Bari Doab ..	—	6,750	194·3	152	8·10
Upper Chenab	0	11,634	191·0	132	7·86
Upper Chenab .. .. Lower Bari Doab ..	0	11,634	325·3	—	—
Upper Chenab	17·2	11,472	173·8	121	7·53
Upper Chenab .. .. Lower Bari Doab ..	17·2	11,472	308·1	—	—
Gujrat Branch, Upper Jhelum	0	2,010	48·0	49	5·3
Main Line, Upper Jhelum	24·6	8,380	73·2	211	9·5
Total, Upper Jhelum ..	24·6	8,380	121·2	—	—
Triple Canal	Below Gauging sites	—	429·3	—	—
Triple Canal, excluding Main Line, Upper Jhelum	—	—	356·1	—	—

TABLE XXI (*continued*)

AVERAGES OF EACH CANAL SYSTEM			RATE OF ABSORPTION LOSS			
Full Wetted Perimeter (feet)	Coefficient of Absorption (*c*)	Total Loss by Absorption under Full Supply (cusecs)	Per million Square Feet of Reduced Wetted Surface	Per Canal Mile	Per cent of Full Capacity	Per Million Square Feet of Wetted Surface
7	8	9	10	11	12	13
76	1·61	1,886	11·42	3·91	23·6	9·88
67	1·64	1,559	11·41	3·42	20·3	9·82
81	1·70	1,327	10·70	3·90	20·4	9·50
98	1·52	2,119	11·26	5·00	19·3	10·10
94	1·52	1,990	11·20	4·75	18·5	10·10
76	1·66	780	9·96	3·52	19·5	8·96
—	—	656	9·70	3·15	16·7	8·73
172	1·31	1,151	10·61	8·57	17·0	0·00
153	1·17	1,726	12·89	11·36	14·8	12·00
—	—	2,877	11·87	8·84	24·7	11·10
141	1·25	1,564	14·00	9·00	13·5	13·00
—	—	2,715	12·32	9·00	23·7	11·52
61	1·83	131	9·55	2·70	6·5	8·6
238	1·18	799	11·25	10·90	9·5	10·6
—	—	930	11·04	7·67	11·1	10·3
—	—	3,645	11·96	8·5	—	11·1
—	—	2,846	12·2	8·0	—	11·33

in their book on Irrigation Engineering, propose the following formula as representing the results of the existing data, to be used in estimating seepage to be expected from contemplated canals:

$$S = c\sqrt[3]{d}\,\frac{P \times L}{4000 + 2000\sqrt{v}},$$

where S = seepage in cubic feet per second.
C = coefficient dependent on the material of the canal.
d = mean depth of water in feet.
P = wetted perimeter in feet.
L = length of canal in feet.
V = mean velocity of water in canal.

Then $C = 12$ for clay soil.
$C = 15$ for clay loam soil.
$C = 20$ for medium loam.
$C = 25$ for sandy loam.
$C = 30$ for coarse sandy loam.
$C = 40$ for fine sand.
$C = 50$ for medium sand.
$C = 70$ for coarse sand and gravel.

When dealing with South African conditions it must be borne in mind that the Indian and American canal systems, on which the above formulæ have been deduced, are of a highly developed character, ramifying over very large areas of level land, and extending, sometimes, great distances from the valleys of the rivers from which the canals were originally diverted.

In South Africa, owing to peculiar topographical conditions, the canal systems are, in comparison, very small, and in most cases they are confined to the sharply defined valleys of their origin, gaining only sufficient height to command the alluvial land deposited in the valley. In the Cape Province, for instance, the canals are made to run, almost throughout their length, along the margins of these valleys, through coarse, sandy soils, or fissured rock, and, speaking generally, the main canals are aligned throughout in very porous material.

It is only in the case of branch canals and distributaries that conditions approaching those of the Indian minor distributaries and water courses are met with. As a result of these circumstances absorption losses in South African canals, unless these are artificially lined, are exceedingly high and often excessive.

Canal losses are enormous even in the case of well designed and efficiently maintained canal systems.

The first systematic experiments were carried out at Rurki Engineering College in 1871. The subject was further dealt with by Mr. J. S. Beresford in 1875, and by Mr. R. G. Kennedy in 1883 (Punjab Irrigation Branch Papers No. 10).

In the Indian experiments the rate of absorption was determined by c being the depth in feet absorbed in one hour. This also includes the evaporation, which, however, is only a small fraction of the absorption.

Kennedy found that on the Bari Doab Canal the mean values for c were as follow:

On watercourses or laterals	0·034
On distributaries	0·012
On main canals and branches	0·008

He estimated that the losses on watercourses alone amounted to nearly 30 per cent of the supply entering them, and that taking the canal system as a whole 47 per cent of the head discharges was lost before entering the fields to be irrigated. When it is remembered that the five largest canals in the Punjab have an aggregate full supply capacity of 41,000 cusecs, the two largest carrying 11,000 and 11,680 cusecs respectively, it will be realized what a loss of nearly 50 per cent of the canal discharge means.

CANAL LINING

During the past twenty years considerable attention has been devoted in India and the United States of America to the question of the prevention of seepage losses by lining canals, distributaries, and laterals or watercourses with some impervious material. Apart from the prevention of losses of water other reasons have necessitated action being taken in this direction, such as avoidance of sloughing of the lower banks of canals on side long ground, prevention of damage to adjoining valuable arable land by waterlogging, the prevention of erosion and the reduction of friction, and therefore the reduction of the canal section in rock cuttings, tunnels, etc.

On a small scale such linings have been common for many years past where water is scarce and very valuable, but their adoption on a large scale is of very recent growth.

In 1903, in the Punjab, India, at a time when great irrigation developments were taking place, experiments on a very large scale were undertaken to test the effects of various linings in

small irrigation channels, the first series of experiments being made by the author, working under the general instructions of the late Mr. R. G. Kennedy, C.I.E., then Chief Engineer for Irrigation Works in the Punjab. The methods adopted and the results obtained are fully set forth in No. 11 of the Punjab Irrigation Board Branch Papers.

The linings used in these experiments consisted of crude petroleum, hydraulic lime slurry, cement slurry, and clay puddle and coal tar. The results were expressed in terms of efficiencies as compared with adjoining unlined channels or controls. Crude oil gave the best results. These lining experiments were continued at various places in the Punjab during subsequent years, and have been reported on from time to time, the last summary being that of Mr. F. W. Schoenemann, Superintending Engineer in 1911 (*vide* Punjab Irrigation Papers No. 11, C., experiments of 1908-1910).

With the exception of the clay puddle the linings were very thin, and in order to preserve them they were covered to a depth of from six to eight inches with soil. The main uncertainty of these experiments was as to the lasting qualities of the various linings, and, in the case of watercourses, as to their immunity from damage of cattle, silt clearances, etc.

With linings four or five years old the average efficiencies, as expressed by the ratio of the loss in unlined channels to the loss in lined channels, were as follow:

Crude oil $\frac{1}{16}$ inch thick	4·0
Clay puddle, 6 inches thick	4·0
Cement $\frac{1}{24}$ to $\frac{1}{2}$ inch thick	4·9

The lasting qualities of oil are very doubtful, and with clay puddle the main trouble is that with channels sometimes dry for long periods of time it is very difficult to prevent drying out and cracking.

In 1913 linings of these materials were tried on one of the big branch canals. A lining of mortar one and a half inches thick, composed of one part cement and five parts washed sand, gave the best results.

Similar experiments were carried out in the United States about the same time as in the Punjab, using similar materials, but generally in thicker layers.

Linings have been adopted in North America in practice on a much larger scale than elsewhere. Taking the seepage formulæ already quoted, viz:

$$S = c\sqrt[3]{d}\,\frac{P \times L}{4000 + 2000\sqrt{v}},$$

Wilson and Davis give the following values of C for lined canals:

$C =$ 1 for concretes 3 to 4 inches thick.
$C =$ 4 for clay puddle 6 inches thick.
$C =$ 5 for thick coat of crude oil (new).
$C =$ 6 for cement plaster 1 inch thick.
$C =$ 8 for clay puddle 3 inches thick.
$C =$ 10 for thin oil or cement grout.

From experiments so far gained in practical working it is doubtful whether any linings other than concrete are efficient and lasting, or will stand up to the ordinary wear and tear.

So far as South African conditions are concerned concrete linings have alone given satisfactory results.

Some valuable information is given by Mr. R. C. E. Weber, of the U.S. Reclamation Service ("Eng. News-Record," Vol. 88, No. 11, page 436), regarding a concrete lining 1½ inches thick placed on 64 miles of the Main Distribution System of the Orland Project. The mixture used was 1 to 7, cost 39 cents per square yard, and lining operations were commenced in 1911.

The losses are expressed in terms of depth of water in feet per twenty-four hours over the wetted perimeter, and the following comparative figures of losses are given for different classes of soil, etc.

In sand and gravel	2·79 feet.
In gravelly loam	2·36 feet.
In loam	1·6 feet.
In gravelly clay	1·6 feet.
In clay	1·53 feet.
1½ inch concrete lining	0·28 feet.

The efficiency ratio is therefore 5·7 in clay soils, and 10 in the case of sand and gravel soils. Apart from the saving in water, the effect of the lining has been to reduce the maintenance costs by 60 per cent.

While concrete linings are undoubtedly the most efficient, it is essential that the materials used and the workmanship be of the best, and close attention must be paid to construction details. Poor work is both costly and inefficient.

CHAPTER VI

CONSERVATION OF WATER

In order to make the best use of intermittent or varying supplies of water running off from a given catchment, reservoirs are created to regulate the flow in such a manner that it can be accommodated to requirements for town supplies, irrigation, or for water power. South Africa, like most warm and dry countries, is a land of intermittent rivers, or where these are perennial in the sense that they are never dry, the fluctuations in the rate of flow at different times of the year are enormous. Thus many of the biggest perennial rivers in the Transvaal, and likewise the great Orange and Vaal rivers, frequently nearly cease running during the critical early spring months of August, September, and October. On the other hand, during rainy periods rivers are often in high flood, and only a very small fraction of the flow is capable of beneficial use, the remainder being discharged into the sea. Furthermore, especially in the more arid regions of the Cape and Orange Free State provinces, the annual run-off varies greatly from year to year, and, so far as is possible, it is necessary to balance the flow in a year of good run-off against a year of bad run-off.

It may be said without question that in South Africa irrigation developed without the aid of conservation works, excepting on a very small scale, is impossible both absolutely and relatively in proportion to the maximum theoretical achievement.

Large conservation works demand, however, considerable assistance from nature, and without favourable topographical and geological conditions the construction of such works for irrigation or water power is not economically feasible. In the case of town supplies and also in the case of small reservoirs for special purposes topographical conditions are of less importance and are frequently ignored altogether, but with such cases we are not here concerned.

Places where storage of water can be obtained at moderate cost are called storage or reservoir sites, and these may be classified as follows :

(1) Natural lake basins.
(2) Favourable portions of natural valleys or drainage lines the flow of which is to be conserved.
(3) Natural depressions away from natural drainage lines, into which water can be led from the catchment under tribute and out of which the water can be led for irrigation or other purpose.

Examples of the first class do not exist in South Africa, but are represented by the Thirlmere and Hawsewater reservoirs of the Manchester Water Supply in England, the regulating works at Lakes Geneva and Zurich in Switzerland, etc. Class (2) includes the majority of storage works throughout the world. These consist either of solid dams across valleys, or of arcades of large sluice-gates across rivers, called "barrages" or movable dams.

The third class has very few existing representatives. Schemes involving the use of depression or "pans" have been under investigation in South Africa, and on a small scale are likely to materialize in the future.

It is intended here only to deal with storage sites of the second class.

In choosing a site for a reservoir four main conditions must be complied with :

Firstly, the contour of the reservoir basin must be such that the storage capacity will be great in comparison with the height of the wall or embankment, which must be built to effect the storage of water.

Secondly, there must be an adequate run-off to fill the reservoir.

Thirdly, the site at which it is proposed to build the dam must be of such a nature that an efficient watertight structure can be constructed at reasonable cost.

And fourthly, it must be possible to discharge surplus water from the reservoir both safely and economically, so as to ensure the safety of the works under the most adverse conditions likely to occur.

The best site for a dam in the country is useless unless associated with an efficient reservoir basin, and vice versa.

Sites which are ideal in all these respects are very rare, and so far as South Africa is concerned they yet remain to be discovered. Some of the finest storage sites have been found to

exist in places where the run-off which can be made available is insufficient to justify the construction of a conservation work. South Africa is, as a matter of fact, singularly deficient in even moderately good storage sites. The selection of a site is invariably the result of a compromise.

Conservation works are invariably costly structures both as regards actual outlay and in relation to the duty they are required to perform. The selection of a storage site throws, therefore, a very heavy responsibility on the engineer. When storage is desired in a given region it is necessary to make the most thorough examination of all the surrounding country lying at a sufficient altitude to serve the needs of the project in question. It must have a sufficient drainage area above it to ensure an adequate supply of water.

A rough reconnaissance survey will generally suffice to bring to notice all the storage sites which may possibly meet the requirements of the case. From amongst these should then be selected those sites which, on further preliminary investigation, promise to fulfil the four main conditions previously set forth, and these should then be surveyed in detail.

In making a final choice of site, apart from the questions of cost and general efficiency of the works, the differences in the run-off yield at the different sites must be taken into careful consideration, and an endeavour should always be made to locate the reservoir as near as possible to the area or point at which the water is to be used.

If the distance is great the transit losses are apt to become abnormally high, and owing to the time taken to pass down the water to the point where it is used regulation becomes difficult.

The efficiency of a reservoir basin is measured by the volume of water impounded at various levels. To illustrate this relationship in actual practice, a number of capacity curves are shown on the annexed diagram (Fig. 14) giving, for a number of reservoirs in South Africa and elsewhere, the capacity in acre feet at different levels.

The South African examples are the following:

Brand Vlei (Worcester District).
Kamanassie (Oudtshoorn).
Van Ryneveld's Pass (Graff Reinet).
Lake Mentz (Jansenville).
Grass Ridge (Cradock).
Hartebeest Poort (Pretoria).

These reservoir dams are all complete or under construction at the present time.

The Burrin Juck Reservoir is in Australia, and the Arrowrock, Rooseveld, and Elephant Butte dams in the western United States of America.

It will be seen from a study of these curves that in most cases, after a certain depth of water is attained, the rate of increase in the capacity is practically the same in all cases, the curves running parallel to one another. There is a very great difference, however, in the respective depths at which this

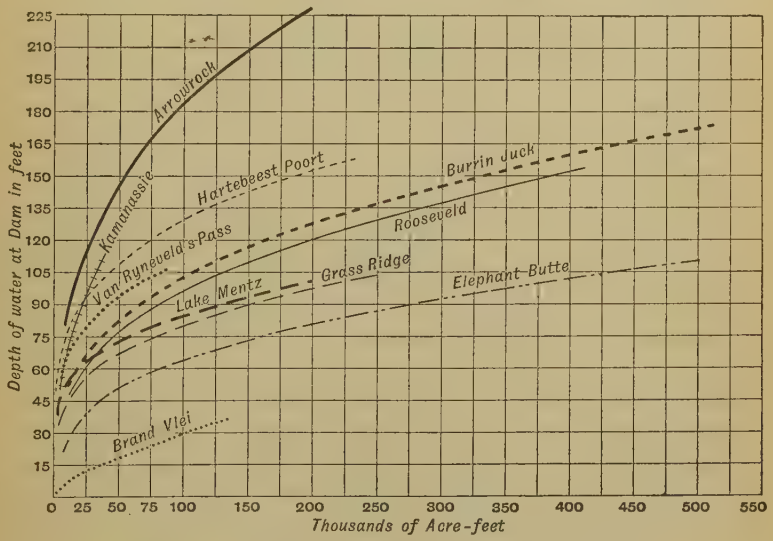


FIG. 14.—Capacity curves of various reservoirs.

parallelism becomes apparent. Each curve commences nearly vertical, and at a certain depth it moves outwards, becoming increasingly flatter but never becoming horizontal. The sooner the curve commences to turn outwards and the sharper the turn the more efficient is the reservoir basin.

The curves in the diagram show, for example, that the Brand Vlei basin is extraordinarily efficient, whereas the Arrowrock and Kamanassie basins are pronouncedly inefficient.

It is difficult to compare, on the same diagram, small and large reservoirs unless logarithmic scales are adopted, and for that reason Table XXII of comparative capacities of thirteen

reservoirs, showing the respective capacities at twenty-five feet intervals of depth is given. Of the examples selected the first nine are in the Union of South Africa.

In practice most reservoirs are small when compared with works such as Burrin Juck in Australia, or Rooseveld and Elephant Butte in the United States. Either the water supply available is insufficient to justify a high dam or topographical or financial considerations impose definite limits to the height of the dam even if water is available. The full supply depth

TABLE XXII

Capacity in Acre Feet of Various South African and other Reservoirs at Different Levels Compared.

Depth of Water in Feet	Calitzdorp, S.A.	Kamanassie, S.A.	Bon-Accord, S.A.	Grass Ridge, S.A.	Tarka, S.A.	Van Ryneveld's Pass, S.A.
25	—	500	900	570	100	—
50	900	5,250	9,525	15,600	4,500	4,300
75	2,500	15,000	32,000	78,000	22,500	21,000
100	6,000	31,100	—	213,000	59,250	65,000
125	—	—	—	—	—	—
150	—	—	—	—	—	—

TABLE XXII (*Continued*)

Depth of water in Feet	Lake Mentz, S.A.	Harte-beest Poort, S.A.	Brand Vlei, S.A.	Burrin Juck, Australia	Arrow-rock, Idaho	Roose-veld, Arizona	Elephant Butte, New Mexico
25	86	230	78,396	—	—	—	4,825
50	9,000	1,836	—	10,000	3,700	11,240	43,350
75	61,000	9,200	—	35,000	—	50,000	169,000
100	185,000	34,340	—	95,000	18,000	108,800	385,000
125	—	87,720	—	185,000	—	230,000	713,515
150	—	187,000	—	340,000	55,600	405,000	1,223,000

of water in most South African dams ranges from about 60 feet to 100 feet. The highest dam so far constructed in South Africa is the Hartebeest Poort dam near Pretoria, which has a full supply depth of 139 feet and a high flood depth of 154 feet.

It is therefore of main importance to aim at the greatest capacity efficiency at these low levels.

It will be seen from the table that with the exception of Brand Vlei, which is an abnormal case, the storage in the first

25 feet is very small. This usually represents the storage in the channel of the river only. Above this depth a rapid increase of capacity with depth is to be desired. It will be noted, however, that with the exception of the Brand Vlei and Elephant Butte reservoirs the desired expansion only takes place near the 50-foot depth or above it. Thus at 50 feet depth the Calitzdorp reservoir only contains 900 acre feet (this is a very inefficient basin), whereas the Grass Ridge reservoir contains 15,600 acre feet at this level. The latter may be considered as a most satisfactory basin.

For South African conditions any basin giving a curve falling inside of the Van Ryneveld's Pass curve, that is, to the right of it, must, in the case of the larger reservoirs, be considered satisfactory.

When alternative reservoir sites have been surveyed the capacity curves should be at once plotted. Such curves not only reveal the comparative merits of different reservoirs but assist materially in determining the position of the outlet, and in cases where scour sluices are installed of the best position for such sluices. When making the preliminary reconnaissance for reservoir sites the first thing to determine is the longitudinal slope of the valley. It is possible to obtain a good idea of the cross sectional slopes by eye, but the longitudinal slopes should be computed by means of an "Abney" level, a good aneroid, or, better still, by actual levelling.

Strange (Indian Storage Reservoirs) classifies in respect to storage capacity, a slope of less than 10 feet per mile as good, between 10 and 20 feet per mile as fair, and exceeding 20 feet per mile as poor, but in all cases the effect of the longitudinal slope may be modified by the nature of the side slopes and the existence of tributary basins.

There are, of course, many cases where no freedom of choice exists, that is to say, there is only one possible site, and the best must be made of such conditions as exist. More often, however, there is considerable choice of sites, and in such cases the character of the basin plays a great part in the final selection of the site.

The accurate survey of a reservoir basin is a lengthy, tedious, and costly undertaking. Such surveys are sometimes done by taking a number of cross sections at right angles to the longitudinal axis of the proposed reservoir and then drawing the contours on the basis of the information so obtained. A more satisfactory method is to fix the levels for the contour plan by

tacheometrical traverses. A main traverse should be run along the lowest line of the reservoir site and others at higher levels. Successive traverses should be linked together by careful levelling.

This method is the one employed in South Africa. An accurate survey by the means of such traverses is essential in the case of the high flood contour, full supply contour, outlet sill level contour, and at certain intermediate levels. The question arises: How many such intermediate contours should be surveyed? In the case of a large reservoir, surveyed contours at even ten foot intervals may involve a vast amount of work. The safe distance between surveyed contours depends upon the accuracy with which the storage capacity at intermediate contours can be interpolated. In this connexion Captain A. ff. Garrett, R.E., in a Government of India official publication published in 1911 and entitled "The General Theory of the Storage Capacity and Flood Regulation of Reservoirs," has developed certain general equations for contour areas and capacities which are very valuable and which are here briefly extracted.

Calculation of Reservoir Areas and Capacities

Let y = height of water surface in feet above any assumed datum.

Ay = area of contour in square feet at y.

Then assume that

$$Ay = a + by + cy^2 + \ldots\ldots + my^{n-1} \ldots\ldots\ldots\ldots (1)$$

where $a, b, c, \ldots\ldots m$ are constants.

From actual surveys the areas of any required number of contours can be measured, and thus the areas $A_o, A_1, A_2 \ldots\ldots A_{n-1}$ corresponding to levels $o, y_1, y_2, \ldots\ldots y_{n-1}$ are found. Substituting these values in equation (1) we have:

$$A_o = a,$$
$$A_1 = a + by_1 + cy_1^2 + \ldots\ldots\ldots\ldots + my_1^{n-1},$$
$$A_2 = a + by_2 + cy_2^2 + \ldots\ldots\ldots\ldots + my_2^{n-1},$$
$$A_{n-1} = a + by_{n-1} + cy^2_{n-1} + \ldots\ldots\ldots\ldots + my^{n-1}_{n-1},$$

and we have n simultaneous equations to determine n constants $a, b, c, \ldots\ldots m$.

Hence we have generally:

$$Ay = a + by + cy^2 + \ldots\ldots$$

as equations to determine the area of any contour at level y, where a, b, c, etc., are easily computed constants.

The cubic contents of a film of thickness dy are Ady, where A is the area of the film.

Hence Cy = capacity of reservoir in cubic feet from zero up to y

$$= \int Aydy \text{ ; or}$$

$$Cy = \int (a+by+cy^2+.....)dy$$

$$= ay+\tfrac{1}{2}by^2+\tfrac{1}{3}cy^3+\tfrac{1}{4}dy^4+..... +k \quad \text{.....(2)}$$

when k is a constant.

The application of equations (1) and (2) is best illustrated by an example.

Suppose that surveys give the following data:

At 932 contour 7·29 million square feet,
At 940 „ 19·34 „ „
At 949 „ 40·20 „ „

and the capacity of the reservoir up to the 932 contour is found to be 12·08 million cubic feet. It is required to find area of any contour and the capacity of the reservoir up to any height. In equation (1) y is the height above R.L. 932. Then for R.L. 932, by substituting for y the values of 8 and 17 respectively,

$$A_o = a = 7{\cdot}29$$
$$a+8b+64c = 19{\cdot}34.$$
$$a+17b+289c = 40{\cdot}20.$$

Solving these equations we obtain:

$$a = 7{\cdot}29,\ b = 1{\cdot}124,\ c = 0{\cdot}04778.$$

Hence the area equation becomes:

$$Ay = 7{\cdot}29+1{\cdot}124y+0{\cdot}04778y^2.$$

Integrating we obtain:

$$Cy = 7{\cdot}29y+\tfrac{1}{2}(1{\cdot}124y^2)+\tfrac{1}{3}(0{\cdot}04778y^3)+k.$$

The constant k is obviously the capacity of the reservoir up to R.L. 932, and is 12·08. Hence:

$$Cy = 12{\cdot}08+7{\cdot}29y+0{\cdot}562y^2+0{\cdot}01593y^3,$$

and if we substitute any numerical value for y the value of Cy is obtained in millions of cubic feet.

This method was checked by applying it to a large number of cases in which capacities at different levels had been determined by the usual calculations based upon surveyed contour areas, and it was found that the error was rarely more than three per cent and generally less. It appears, therefore, that practically as accurate results can be obtained from the measurement of three contours as from nine contours.

When it is borne in mind that reservoir contour levels are in most cases based upon cross sections or traverses and spot

levels taken with a tacheometer, and so depending upon the judgment of the draftsman, the errors become of even less importance, and for most purposes it would seem to be sufficient if three contours were carefully surveyed in the case of small reservoirs and five in the case of large ones, and intermediate contours calculated as suggested.

The selection of a good site for a reservoir basin presumes the existence of a satisfactory site for a dam by means of which the reservoir will be created. The location of such a site is a matter of considerable difficulty and requires experience and judgment.

A reservoir dam is a structure which by forming an artificial barrier across a valley, or generally across a drainage line, will hold up the water behind it and thus form a lake or reservoir from which the water stored can be withdrawn as desired. To serve its purpose satisfactorily a dam must be stable under all conditions likely to obtain and it must form, practically speaking, an absolutely watertight barrier. Failure to comply with this requirement, apart from causing the loss of valuable water, will in most cases materially affect the stability of the structure. A leaky dam is generally an unsafe structure likely to fail.

Watertightness in this case is not confined to the artificial structure itself, but more especially to its foundations and abutments. While fulfilling the above requirements, the dam must not be abnormally costly.

This question of cost is generally the determining factor. Engineering skill and experience is capable of overcoming most difficulties imposed by unfavourable conditions of the site, but unless a unit quantity of water can be stored and made available within a certain amount of capital expenditure the construction of the dam is not practicable.

The topography and geology of the site are therefore equally important.

Assuming the existence of an efficient reservoir site, that is, that the capacity of the reservoir will be great compared with the height of the dam, the site should be as narrow as possible, and unless conditions are such that it is necessary and practicable to pass the surplus flood water over the crest of the dam the local topographical conditions must either provide a natural site for a spillway or waste weir, or must allow of such a spillway being created at reasonable cost.

Other important conditions are :

The availability and character of materials for construction.

This frequently determines the character of the design of the dam, i.e. whether it shall be a masonry dam, an earth dam, or a rock-fill dam, or other type.

The requirements of construction, such as the disposal of the normal and flood flow whilst the dam is being built. These have an important bearing on the type of design adopted.

The existence of suitable sites close to the works for construction plant, store yards, camps, etc., and finally accessibility of the site and transport facilities. These very materially affect the cost of the works.

A survey for the location of a dam thus not only necessitates the very detailed contouring of the site and accurate cross sections thereof, but also a detailed geological survey to determine the true nature of the foundations and abutments of the proposed dam and the nature of the reservoir basin, and likewise to reveal the character of the building materials available, such as rock for masonry or concrete ballast, gravel, sand, etc.

In the first place, the general geological conditions of the site should be studied to ascertain the strata underlying and flanking the proposed reservoir and site for the dam. The geological structure may contribute to the efficiency of the reservoir, or it may be so unfavourable as to render the project impossible. A reservoir site in a synclinal valley is generally favourable, whereas an anticlinal valley is often unfavourable. In the latter case the strata dip away from the reservoir and may give rise to serious leakage losses.

When extensive igneous sheets form the floor of a reservoir, great caution must be exercised, as such sheets frequently contain crevices and caverns through which water may escape. Such conditions have so far not come to notice in South Africa, but have given rise to considerable trouble in some of the western States of America. As regards leakage the most treacherous formation in South Africa is the dolomite, which is represented largely in the Transvaal. There is also a narrow band of this formation at the foot of the Zwaartberg Range in the Oudtshoorn district of the Cape province. The leakage through this formation is so great that it is not safe to locate a reservoir on any portion of it.

Reservoirs located in the contorted and crumpled quartzites of the great coastal ranges along the southern margin of South Africa are also liable to leakage, but such leakage is sometimes

amenable to treatment, as was the case with the Prins River reservoir in the Ladysmith district of the Cape province.

The foundations and flanks of a dam form generally the main sources of anxiety.

In this connexion general geological investigations and considerations are extremely valuable up to a point. From these alone it may be possible to reject a site as useless, or it may be possible to pick out a number of sites or one or more lengths of the valley or gorge at or within which the final selection must be made.

It is very rarely the case that a final location of the dam can be made without making test pits, cuts into the hillsides, or borings. Expert geological knowledge of the site will in many cases obviate the waste of much money in boring or excavating pits at sites the shortcomings of which might be predicted.

It is very rarely the case in South Africa that sound rock is exposed at a dam site extending right across the bed of a river and on its flanks. Such exposures are frequent enough along the course of the rivers, but they would seldom be selected as sites for dams, as in practically all these cases the gradient of the bed will be found to be very steep, and this again indicates an uneconomical storage site.

In South Africa, as elsewhere, dams are located at points along a river where a massive rocky ridge is cut through, or in gorges. Such contractions of the valley, whether only a few hundred feet in length or forming gorges extending sometimes for miles, are known in South Africa as " poorts."

Generally these are cut through very hard rock, and form a barrier which causes the river gradient above it to be comparatively flat, whereas the channel through the poort is steep.

It is seldom the case that the narrowest part of the gorge can be selected, and most sites are located as near the upstream end of the poort as possible. At such sites the rock is generally only visible on the flanks at higher levels, the foundations and lower flanks being covered by alluvium and wash from above.

In the Karroo, i.e. on the plateau area of the Cape and Orange Free State provinces, the most common sites for dams are at points where the rivers break through defined ridges formed by dolerite dykes or sheets of dolerite which have indurated the sandstone or shales forming the normal country rocks.

These poorts have nearly all gone through the same process of evolution and form a record of evidence of great changes in climate. So far as the dimensions of the gaps in the rock are

concerned, these were attained in a past age when the rainfall was very much greater than at present.

The high run-off of those days scoured deep and wide channels down to bed-rock and kept them comparatively free from silt or gravel.

This wet period was followed by a period of great aridity during which the normal run-off was unable to transport the enormous products of erosion which go with arid conditions. The valleys then became choked with silt and sand, and at the commencement of the historical period rivers in the ordinary sense were few in number. The great drainage lines consisted for the greater part of extensive flats of deep alluvial soil, much of it composed of extremely fine material. At the poorts, where the gradients were steeper, small river channels no doubt continued to exist.

Within the last century various circumstances have combined to initiate a rejuvenation of the ancient river systems, and channels are now being excavated through the alluvial deposits, but as yet it is only rarely that these channels have been cut down to bed-rock.

The conditions obtaining at Kaffir Poort, in the Britstown district, are very typical. About 1909 this site was selected for a large earth dam impounding over 4,000 million cubic feet, which was constructed by the Smartt Syndicate, Ltd., for irrigating purposes. The Ongers River, which is dammed, has here a catchment of over 4,000 square miles. About the year 1860 this river had no defined channel in the neighbourhood of the present dam. Fifty years later, as the result of overstocking of the veld and consequent erosion, the Ongers River had cut a channel about 300 feet wide and from 20 to 25 feet deep through Kaffir Poort and above and below it. This poort is 600 feet wide, and when foundation excavations were made for the concrete core of the dam the ancient river bed was discovered at a depth of about 40 feet below the alluvial benches and some 20 feet below the present bed of the river. The ancient bed was found to be almost level in cross section, about 550 feet wide, and excavated out of very solid dolerite.

In the Salt River Poort in the Beaufort West district the bed of the ancient river was found to be 1,150 feet wide, 48 feet below the present ground surface, and 40 feet below the modern river bed. Conditions of this kind will be found repeated at almost any of the Karroo poorts.

In some cases the ancient bed is found to lie at very great depths below the present levels. In one case borings proved the detritus to be over 100 feet in depth and in another case 80 feet deep.

In all such cases borings are essential to prove the nature of the foundations.

In South Africa geological conditions are generally so uniform that comparatively few borings will reveal the true nature of the rock bottom.

In a paper on " Geology in Dam Construction " read by Dr. Alex. du Toit, D.Sc., F.G.S., Geologist to the Irrigation Department, before the South African Society of Civil Engineers on April 12, 1922, much valuable information concerning the geology of dam sites in South Africa is given, and also much sound advice regarding exploratory borings, derived from experience gained by the Irrigation Department in the course of the past fifteen years, and much of what follows concerning these matters is taken from his able summary.

In the case of relatively unjointed and uncleaved formations, which include most of the horizontally bedded Karroo strata, the rocks beneath the alluvial convering are generally in sound condition, and all that is required is the depth at which they lie and their lithological characters. In these circumstances it is unnecessary to bring up a core or continuous sample of the underlying rock, and it will be sufficient to probe with long iron rods or jumpers, sink trial pits, or drill a series of holes with a percussion drill, according to the depth of the overlying wash or alluvium.

In some cases, especially where earth dams are under consideration, and the earth covering is free from large boulders, an earth auger can be used with advantage.

In all cases where the character of the underlying rock is not definitely known, or when the rock is jointed, has a slaty cleavage, or is irregularly weathered, it is necessary to prospect with a drill which will produce a core, such as a diamond drill or a chilled shot drill.

A core drill works badly in loose material, and when a considerable depth of such material exists it is now the invariable practice in South Africa to drill holes through the loose ground down to the underlying rock with a jumper or percussion drill, casing the hole throughout. The rotary core drilling is then carried on inside the casing. The cased hole is usually six inches in diameter. The hole in the rock is of small diameter, and in

the Union the diameter of the core is generally about $1\frac{3}{8}$ inches. It is necessary to carry these holes to a considerable depth sometimes. In the case of dolerite, which forms the foundation of many South African dams, it is frequently difficult to distinguish between the solid dyke rock and large boulders or masses of sound rock surrounded by partially decomposed dolerite.

The inclination of the bedding planes must be carefully noted, as if there is any variation between the core and exposed strata on the flanks the disturbed nature of the formation is then revealed.

It must not be imagined that cores will always produce clear and reliable evidence of the true nature of the underlying rock, and in this connexion the following extracts from Dr. du Toit's paper is of importance.

The difficulty of interpreting the evidence supplied by cores obtained from the solid applies more particularly to the basic igneous rocks in which weathering follows the joints, so that crumbly material can be struck after passing through several feet of fresh and apparently sound rock, and perhaps more than once.

Bleaching or else staining along the joints are useful indications, but as depth increases the faces may be quite fresh and yet belong to open cracks.

At greater depths where the joints become more and more potential it becomes necessary to decide whether the fractures in the core are natural or have been caused in the process of cutting or extraction.

All contacts between dissimilar rocks, such as dolerite and shale, require the most careful study.

In much jointed rock contiguous holes will give widely discordant results, so that the degree to which the formation as a whole is weathered, and consequently the precise depths to which excavations for foundations will have to be carried, may still be somewhat uncertain.

However, by increasing the number of test holes a fair indication can always be gathered, while an indication of the relative watertightness of the rock will be furnished by the quantity of water progressively lost during boring.

Where poorts or gorges are narrow and cut in hard material, such as quartzite, the rocks of the floor will only in the rarest of instances be less sound than that exposed at water level, so that it will only be necessary to corroborate probings with a bar or jumper by one or two diamond drill holes.

So far as South African conditions are concerned there is a

great deal of interesting and important matter in Dr. du Toit's paper to which the engineer in South Africa should pay close attention.

In all cases when important structures of this kind are projected the engineer is advised to work in collaboration with a competent geologist who is familiar with the local conditions.

The number of boreholes drilled at any site is a matter of judgment. Much depends upon the site. Sometimes one or two holes will suffice, in other cases a large number is necessary. In narrow gorges a chain of holes may be required down the longitudinal axis of the channel.

In other cases one or more rows of boreholes on successive cross sections are desirable. In all cases it is advisable to stagger the holes so as to cover the greatest area of floor. By way of example it may be stated that in the case of several important South African dams, constructed or projected, the numbers of test boreholes were as follows:

TABLE XXIII

Exploratory Boreholes Drilled in Connexion with Certain South African Irrigation Projects.

Name of Dam	District	Type of Dam	Description of Foundations	No. of Holes	Aggregate Footage (ft.)	Shot or diamond drill Footage
Grassridge	Cradock	Earth	Decomposed dolerite and shales	4	142	—
Tarka	Cradock	Rock-fill or concrete	Dolerite and shales	6	225	—
Van Ryneveld Pass	Graaff Reinet	Rock-fill or concrete	Dolerite	9	392	137
Waterval	Bloemfontein	Earth	Sandstones and shales	23	919	—
Hartebeest Poort	Pretoria	Concrete	Quartzite	10	Approx. 700	—
Kamanassie	Oudtshoorn	,,	Greywacke and shale	10	599	481
Lake Mentz	Jansenville	,,	Glacial conglomerate	17	Approx. 600	200
Beer Vlei	Willowmere	Earth	Quartzite and shales	18	889	236

Deep borings into the rock show approximately how much thereof will have to be excavated in order to obtain a reliable foundation, i.e. one which will be watertight and will carry with safety the load of the dam when supporting the maximum head of water. This is, however, not always the case, as no matter how many test holes are drilled, fissures and other undesirable occurrences may escape detection. Such was the case, for example, at Calitzdorp, where, in spite of a very liberal allowance of boreholes, a local fissure necessitated the carrying down of the foundation to a depth of fifty feet in one place, or about thirty feet lower than anticipated. At Lake Mentz, owing to the dipping of the strata downstream at a higher angle than anticipated and to a very small alteration in the location of the dam, the foundations were throughout deeper than estimated.

In some cases it is necessary to carry the foundations to a very great depth in order to obtain thoroughly satisfactory conditions. Thus in the case of the Howden and Derwent dams of the Derwent Valley waterworks in Yorkshire, England, foundations, including concrete cut-off walls, were carried to maximum depths of 190 feet and 98 feet respectively, the greater part of these excavations being in rock.

Rock excavation of this kind is exceedingly costly, as blasting operations are usually permissible on a limited scale only, or are entirely barred.

Where the foundation rock is generally sound and fit to carry the load, but contains open joints, an attempt to reach a thoroughly sound bed by excavation alone is in many cases not justified, and equally good results can be obtained by forcing cement grout into the joints under pressure.

This procedure has been adopted on a large scale at the Lake Mentz dam. The hard, slaty rock formed from a boulder clay of glacial origin was found difficult to excavate, and even the mildest blasting operations caused considerable disturbance in the surrounding sound rock. It was also found that when the fresh hard rock was exposed to the atmosphere it tended to expand and cause joints to open at lower levels. As this process was likely to continue indefinitely it was decided to carry the excavations for the foundations down to hard, unweathered rock only. The concrete structure of the dam was then commenced, leaving a double row of holes across the upstream face of the dam. These holes were carried up to a considerable

height. Holes were subsequently drilled into the bed-rock through these holes in the structure, by means of a diamond drill, the drilling being carried to varying depths according to whether the holes proved to be watertight or not. Cement grout was then forced into the holes and all open joints thereby sealed. Before the cement grout is forced into the holes water pressure should be applied to clean out the holes and cracks. At this stage it is advisable to apply the same head of water as the dam will ultimately be called upon to stand. At Lake Mentz the maximum pressure employed was 100 lbs. per square inch.

Altogether 1,900 feet of grout holes were drilled in this manner. Only moderate pressure must be employed for work of this kind, as otherwise fissures may be actually created.

A full and interesting description of the pressure grouting at the Lake Mentz dam is contained in a paper read before the South African Society of Civil Engineers by Mr. D. Holtzhausen in 1922, entitled "Grouting of Foundations at Lake Mentz."

The above-mentioned process will often be found to be cheaper and more satisfactory than relying upon excavation alone.

Wherever possible a cut-off wall should be provided, even if grouting operations have to be carried below the foundations of the cut-off wall.

In the United States of America pressure grouting has been adopted in many recent dams—for example, the Estacada dam in Oregon, the Lahonton dam and Arrowrock dam, both of the United States Reclamation Service.

In dealing with foundations so far we have largely considered the case of concrete or masonry dams. If conditions are favourable for the construction of an earth dam or rock-fill dam, the problem is sometimes less complicated, as instead of having to provide a satisfactory foundation under the entire seat of the dam which becomes inconveniently wide at any considerable depth, it is now only necessary to provide a good foundation for a narrow core wall or cut-off wall. The material overlying the foundations must, however, especially in the case of an earth dam, be suitable for incorporation in the body of the structure; or if removal is necessary, the mass to be so removed must not be excessive in quantity.

In both these cases, also, satisfactory material for constructing the dam must be available in close proximity to the site. In the case of earth it should preferably be located in the basin of

the reservoir upstream of the dam. For this purpose the ground must be thoroughly prospected, as not only is it frequently necessary to mix different classes of soil to obtain the best material for the upstream half of the dam, but care must be taken not to prejudice the watertightness of the reservoir by piercing a thick layer of clay soil and exposing sandy and more porous layers below.

Having decided on the suitability and economic practicability of the foundations, there are certain geological considerations which must be studied in connexion with the disposal of surplus water. Whether this water is to be passed over a spillway at full supply level or through sluices in the dam, the effect on the works and on the surrounding country generally of discharging enormous volumes of water at very high velocities must be carefully studied. Elaborate and costly works are often necessary to deal with this waste water in such a way that erosion will be minimized, the dam made safe from possible danger, and destruction of land and property be guarded against.

Difficulties in connexion with the disposal of surplus water frequently become the determining factor in the selection of the type of dam to be constructed. In the case of concrete or masonry dams water may sometimes be discharged over the crest; but in the case of earth and rock-fill dams this is *not* permissible.

The height to which a dam shall be built or the depth of water at full supply level depends upon various factors.

In rare instances the available supply is in excess of requirements, and in such cases the dam is made only high enough to impound such portion of the supply as is necessary for the period, being one year or less, during which storage is wanted.

In most cases in South Africa and other dry countries the supply is usually not equal to the demand, and, moreover, it fluctuates greatly from year to year. The problem then becomes one of fixing the capacity at the most economical figure. The reservoir may not fill in certain years, whereas in other years there will be a big surplus. If there were no evaporation and absorption losses, or if these were comparatively small, it would be possible to balance good years against bad ones over a series of years, and thereby make the best use of the water available. Unfortunately this is not the case. As has been already shown the effective storage capacity of a large reservoir is generally above the fifty-foot level, and with evaporation losses ranging

from five to nine feet per annum the storage of water for periods exceeding two years becomes uneconomical in South Africa.

Methods of determining the equalizing capacity of a reservoir on the basis of monthly or yearly mass curves, which are found so useful in temperate and more or less humid countries where the fluctuations in run-off are comparatively small, are inapplicable to conditions existing in South Africa, the western United States, or Australia. Table XII in Chapter III illustrates very clearly how great are the fluctuations in the annual run-off from South African catchments.

Any attempt at completely balancing such fluctuating quantities is manifestly impossible, even if years of exceptionally high discharge are left out of account. In the drier areas of the Union it frequently happens that two years of very poor run-off follow one another, though it is rarely the case that three very bad years succeed one another. It does not, however, follow that a balance would be effected over a three-year period. If very wet and very dry years are eliminated, something more approaching a balance could be attained on a three-year basis, were it economical to carry over storage for so long a period.

In view of the shortness of most of the reliable run-off records, a somewhat arbitrary determination of the full supply capacity is unavoidable. Thus if from a record of a considerable number of years any abnormally wet or abnormally dry years are eliminated, and the mean of each successive two years is calculated, a reasonable value of the available yield could be obtained by adding thirty-three per cent to the lowest average for any two-year period. In order to secure this mean available yield, the capacity of the reservoir should be made from twenty to thirty-three per cent larger than the capacity so obtained. This additional percentage will constitute the maximum carry-over from one year to the next.

This can best be illustrated by an example.

The years 1900 and 1901 were abnormally wet ones, and 1908 was quite abnormally dry.

The arithmetic mean for the whole 23-year period is 1,844 million cubic feet, or if the three very abnormal years are eliminated, the mean becomes 1,724 million cubic feet.

If we consider the 3-year averages the range is still from 33 per cent above the modified mean to 26 below it.

If we eliminate from the 2-year averages those including any

of the abnormal years, the lowest average is 1,160 millions for the years 1914 and 1915. Adding 33 per cent to this we get 1,547 millions as the "working" yield. To secure this the reservoir capacity should be made 20 per cent larger, viz., 1,856 million cubic feet.

TABLE XXIV

Yields from Catchment in Millions of Cubic Feet.

Year	Computed Run-off	Averages for Successive Three Years	Averages for Successive Two Years
1895	1,750	—	—
1896	2,680	—	2,215
1897	620	1,683	1,650
1898	2,650	1,983	1,635
1899	2,120	1,797	2,385
1900	4,232	3,000	3,175
1901	3,625	3,325	3,928
1902	1,737	3,197	2,681
1903	850	2,070	1,293
1904	1,500	1,362	1,175
1905	1,690	1,350	1,595
1906	1,850	1,680	1,770
1907	1,425	1,655	1,638
1908	100	1,125	763
1909	2,680	1,402	1,390
1910	1,260	1,350	1,970
1911	2,930	2,290	2,095
1912	720	1,633	1,825
1913	1,975	1,908	1,348
1914	1,420	1,372	1,698
1915	900	1,432	1,160
1916	1,500	1,273	1,200
1917	2,200	1,463	1,850

Such a supply, viz., 1,856 million cubic feet, would only have been obtained in 10 years out of 23 years.

If the water running into the reservoir is heavily charged with silt and no provision can be made for scouring this out it is advisable to allow a considerable margin for silt deposit. The additional height of dam allowed to give the extra capacity varies with circumstances, depending upon the quantity and character of

silt to be dealt with, and upon the efficiency of the reservoir basin. Thus if a few feet of extra height makes a large difference in the capacity a fairly liberal allowance may be made.

The Hartebeest Poort dam, which is the highest in South Africa, is being built just below the point where the Crocodile River (referred to in Table XII) is gauged. In the original project design the full supply level was fixed at 5,100 millions cubic feet, but it is likely that this may be increased to nearly 6,000 millions cubic feet. The evaporation losses are in this case lower than in the more southern parts of the Union, and the large reserve storage capacity may be justifiable in this case.

The annual draw-off for irrigation should be on the basis of approximately two-thirds the net available capacity of the reservoir if designed according to the rule given above.

In very bad years there will be a shortage of water, and, so far as South African irrigation projects are concerned, this must be faced philosophically by farmers using the water. It is quite certain that if an attempt were made to ensure by storage against the worst years on record irrigation development would have to be limited to a quite unjustifiable degree.

Cases frequently arise when the natural conditions are such that it becomes cheaper to construct a higher dam than is justified by the supply available than to build a lower dam, and provide a very costly spillway which may necessitate an excessive amount of excavation for the purpose, often in solid rock.

Heavy cutting for waste weirs is invariably very costly and is frequently limited as far as possible to the amount of excavation which can be used in the construction of the dam. If no use can be made of the spoil from the excavation it imposes a very heavy financial burden on the scheme.

In some cases reservoirs are made excessively large in order to act as regulators in the case of high floods, which might cause serious damage in the valley below, and to compensate for silt deposit over a long period. Thus in the case of the Elephant Butte dam in New Mexico no less than 400,000 acre feet has been reserved for flood control.

There is a considerable choice of types of dam which may be built to conserve water, and one of the most difficult tasks of the engineer is to decide which particular one to adopt. Local conditions sometimes make the adoption of a particular type imperative. Whenever there is a choice, the problem is to design the cheapest form of structure compatible with safety

and efficiency. The usual types which are applicable to South African conditions are the following:

(1) Earth dams.
(2) Solid masonry or concrete dams of gravity section.
(3) Arched masonry or concrete dams.
(4) Hollow concrete dams.
(5) Rock-fill dams.

Also combinations of several of the above varieties. The best selection for a particular site is largely a matter of judgment, and experienced engineers will often hold diverse opinions on the subject.

It is not intended to deal in detail with the design and construction of all the various types of dams as numerous excellent textbooks are in existence which deal with those matters. The merits or otherwise will be briefly dealt with, with particular reference to South African conditions, and some typical examples of dams of various types actually constructed or suitable for conditions in that country will be given.

In South Africa one important fact must always be borne in mind, namely, the high cost of construction, especially if it involves the use of much skilled labour, or of materials which have to be imported or transported over long distances.

In the case of irrigation schemes the home market for agricultural products grown under them is very limited owing to the smallness of the population, while high transport charges from areas hundreds of miles from the nearest port impose a very severe handicap on the export of such produce. The capital cost of irrigation works is thus in nearly every case the prime factor, and influences more than any other—with the exception of safety—the character of the design and choice of type.

When dealing with reservoir dam designs what Geo. L. Dillman calls the "Great Hydraulic Principle" must ever be borne in mind, that is, to construct one impervious surface and to build the remainder of the structure to support that surface.

This principle is the basis of the design of many modern types, such as rock-fill dams, hollow dams, and masonry or concrete gravity section dams.

The various types of dams will now be briefly dealt with.

EARTH DAMS

Earth dams are generally constructed in one of the following ways:

(1) A homogeneous embankment of earth in which the material is alike throughout.

(2) An embankment with a central core of puddle, concrete, or masonry, with earth on either side of it.

(3) An embankment composed of specially selected earth on the upstream side and second grade material grading through to coarser material on the lower side.

(4) Similar to the previous type, but grading through to rock-fill on the lower side.

(5) Similar to types (3) and (4), but with the addition of a core as described in type (2).

The particular type selected will depend upon the material available locally.

The main advantages of earth dams as compared with other classes are :

(*a*) They require less solid and expensive foundations.

(*b*) They are more easily raised from time to time to increase the storage capacity or to make good any deficiency of storage due to silting.

(*c*) They can be constructed rapidly with unskilled labour and with material available on the spot.

(*d*) They are generally the cheapest form of construction when suitable material is available.

The disadvantages of this type are that they are more exposed to damage by floods during the period of construction ; and as no water must be allowed to flow over the top, special and costly waste weirs must often be constructed.

Earth dams have greatly increased in height in recent years, and, provided the material and construction are good, no particular limit can be fixed. It will generally be found that there is an economic limit, however, and that above a certain height a masonry or concrete dam will be cheaper to construct.

The essential features of an earth dam are as follow :

(*a*) It must be made absolutely secure against any possibility of overtopping by providing efficient ways of disposing of all surplus water either by means of a waste weir or sluices or both, and by the provision of ample free board.

(*b*) The dam taken as a whole must be practically impervious to water.

(*c*) The foundations and abutments must be practically impervious and thoroughly bonded to the dam.

(*d*) The slope of the dam must be protected against erosion by wave action by means of pitching, slabs of concrete, or other efficient devices.

(*e*) The cross section of the dam should be made such that sloughing will not take place, and the downstream half of the dam should be adequately drained.

In the drier parts of South Africa, i.e. in the Cape and southern Free State, excellent material is often available for dam building. The alluvium of all Karroo rivers consists of various grades, ranging from a heavy red pot clay to pure sand. By careful selection and judicious mixing an ideal material can be obtained which consolidates well, does not crack when dry, and is impervious.

With material of this class it becomes quite unnecessary to provide a core of puddle or of concrete in the centre of the dam above the ground surface. A clay puddle core is in fact more likely to do harm than good if it is carried high up into the body of the dam. Hydrographic conditions are such that it becomes inevitable that the reservoir frequently impounds very little water. In the hot and intensely dry atmosphere of the summer months in the dry parts of the Union the drying out and cracking of the upper portion of the puddle core is a very likely contingency.

In the Transvaal and northern Free State the soil in the valley bottom nearly always consists of black turf. This is very inferior material for dam building as it cracks badly on drying and does not bind well. When, as is often the case, it is mixed with calcareous nodules it is quite useless for this purpose. This black soil should be mixed to a large extent with the sandy soil, which is generally found in abundance on the upper margins of the valleys.

Great care must be taken with the foundations. If these consist wholly of alluvial material and, apparently dense and impervious at the surface, they are certain to contain strata of sandy or gravelly material below, which may be naturally exposed in the floor of the reservoir basin or become exposed in the borrow pits from which the earth from the dam is taken. Reliance on such foundations has been the cause of several bad disasters in the Cape province in the past, owing to the water under great pressure bursting through the floor of the reservoir with such sandy layers, and passing beneath the foundation of the dam, causing the collapse of the latter.

The entire prevention of percolation of water through an earth dam is impossible, but it must be rendered so slow and difficult that it is incapable of moving any of the particles of the soil in the dam or in the ground below, and is thus harmless.

Sites for earth dams generally consist of a river channel flanked on one or both sides by alluvial benches or rock. The river channel is either eroded out of the alluvium, and its bed may still be contained in the mass of alluvium, or it may have cut through it down to the rock surface below; or, again, the trough may have become partly refilled with sand, boulders, etc.

Foundations.—An earth dam may be bedded on material of great variety, and the essential requirements are that the ground should be of a compact nature and will not settle to any material extent under the weight of the dam, nor yield or slip when wet. Light, powdery soils lacking in cohesion or which become very soft if wetted are unsuitable. Soil heavily charged with alkali or "brak," more especially with sodium carbonate, must never form the seating for a dam, as such soils lose all cohesion when wet. If the soil is thus impregnated to a moderate depth only the objectionable material should be removed and run to spoil. It should on no account be used in the construction of any portion of the dam.

The dam may be founded on rock if this is compact and unfissured and practically level, so as to obviate any chance of the earth structure sliding on the rock surface. A good, homogeneous, heavy loam is, however, preferable, as a better bond between the dam and its foundations is thus obtained.

Porous material, such as sand, gravel, loose rock, is, generally speaking, inadmissible.

Provided the soil is not highly impregnated with brak salts, or is not composed of calcareous black turf, the soil comprising the bottom of most South African valleys is suitable for the purpose, and this is especially the case in the Karroo districts of the Cape and southern Free State provinces.

All surface soil containing vegetable matter, blown sand, animal burrows, and pockets of unsuitable material must be removed and run to spoil and not embodied in any part of the dam.

The seating should then be cut into a series of benches, twenty or twenty-five feet wide, running parallel to the long axis of the dam. These benches should be tilted slightly towards the centre of the dam.

Before material forming the dam is placed on the site the surface of the benches should be furrowed with a plough.

If the natural surface consists of sand and boulders or similar material overlying solid rock, this should be removed down to rock level, or to suitable compact material on the upstream half of the dam.

Cut-off Wall.—In practically all cases it is necessary to provide a substantial cut-off or core wall, which will be carried down to bed-rock. Such cut-off walls are generally placed along the centre line of the dam.

Cut-off walls may be made of clay puddle or of concrete. Sometimes it is advisable to make the lower portion of the wall of concrete and then continue upwards with clay puddle.

The greatest care must be taken to ensure a perfect junction between the core wall and the rock foundation, and between the puddle and concrete sections of the wall.

Where a core wall is built in sand, gravel, and boulders, the trench should be cut with sloping sides and the spaces between the core wall and the trench sides back filled with the very best dam building material available, placed in layers and well punned.

It is neither necessary nor advisable to carry the core walls more than two or three feet above the surface level of the alluvial bench if the material for dam building is of good quality. The thickness of the core walls varies according to local conditions and with the material used.

Unless the cut-off or core wall is taken down to bed-rock, which is generally the case in South Africa, the depth of the wall depends upon the nature of the subsoil and on the depth of water in the reservoir. It is very seldom necessary in ordinary compact soils to carry it down more than one-half the depth of the full supply in the reservoir.

As the capacity of the water to percolate through the subsoil decreases with depth it is unnecessary to make the width of the core wall uniform throughout its depth. It is usual to taper it in sections, making its width at the bottom less than at the ground surface.

The minimum bottom width is determined by conditions of working. If a good, sound puddle core is to be secured the bottom width, in the case of a large dam, should not be less than six feet. For Indian practice Strange gives the following rules for determining the base width: Either make it one-

quarter of the full supply depth or one-eighth of the full supply depth plus three feet.

The core wall should taper upwards with a batter not steeper than $\frac{1}{4}$ to 1.

The core should be carried to a height of about three feet above the natural surface level of the valley bottom outside the river bed. In the river section it should be carried up to a height of not less than four feet above the level of the alluvial banks of the river, and the wall should be made a little thicker. Above the natural surface level the core wall should again taper, becoming narrower, measuring upwards.

True clay puddle is very seldom obtainable at many dam sites in South Africa, and in such cases the best clay procurable should be used. The less clayey the material is the thicker should be the cut-off wall.

In some cases where the subsoil is pervious, and no clay soil suitable for a good core wall is available, a concrete core wall may be substituted. Concrete core walls are likewise suitable where it is necessary to carry these to great depths.

Such cores should consist of rich concrete and only the best materials should be used.

The width of the wall is generally determined by practical working conditions, such as the depth of the core trench, character of the trench timbering, etc. For deep trenches the width should be about six feet. In the case of shallow depths they may be four or even three feet. Where concrete cores are carried up to a considerable height above ground level, as in the river sections, it is advisable to reinforce them. Great care must be taken to carry up the earthwork uniformly on either side, as if subjected to unequal loading they will crack and become useless as cut-off walls.

Even when good clay puddle is available it is advisable, if the core wall is of considerable depth, to construct a concrete key several feet in height on which the puddle wall will rest. The upper surface of the key wall should be made in the form of a flat "V" so that the puddle will pack tightly on to it.

Section of an Earth Dam.—The section of an earth dam cannot be standardized, as it varies considerably with the nature of the material used in construction, the character of the foundations, and also with the height.

Using the same materials, a dam of low height could be

built with a relatively much smaller section than would be safe with a dam of great height.

So far as possible it is desirable to preserve the character of the section throughout the length of the dam, but where this length is considerable and a relatively large part thereof is of comparatively small height it is sometimes economical to vary the section, making it larger in the higher portions, and especially in the river section, than on the flanks, where the height is small. Assuming that the foundations are good and that an efficient cut-off wall will be provided, the section of the dam depends largely upon the angle of repose of the material when saturated with water. This varies greatly with different soils. If the base width of a dam is made too narrow the structure is liable to fail by sloughing at the downstream toe, owing to the soil particles losing cohesion and the mass becoming semi-liquid.

The behaviour of earth dams when partly saturated was investigated on a large scale in India, more especially in the Bombay Presidency, thirteen dams being experimented on. The results are fully set forth in a Bombay Government Technical Publication on "Results of Experiments to Ascertain the Saturation of High Embankments of Storage Tanks in the Bombay Presidency," 1908.

Similar investigations were made in the United States by a Board of Engineers appointed to consider the proposal to build a part of the Croton dam in earth.

The limits of saturation as plotted on the cross sections form a very irregular line depending upon the location of various classes of material in the dam, the position and nature of core walls, provision for drainage in the downstream half of the dam, etc. The saturation line or hydraulic gradient which starts at the point where the full supply level cuts the upstream face of the dam must never cut the line of the downstream slope above its toe or point of intersection with the ground surface. Experience may be said to have demonstrated that the virtual line of the hydraulic gradient is likely to approach a slope of 1 in 4 in the case of dams built largely with clayey material, or 1 in 3 in the case of dams built with good loamy material in the upstream half and well drained in the lower half.

The deductions drawn from the Bombay experiments by Mr. A. Hill, Chief Engineer, were that it is unsafe to construct a high embankment entirely of clayey material, and that it is

essential to use good drainage material in the downstream part of the dam. The Bombay experiments were undertaken as the result of a number of serious slips which occurred in the case of nine dams, and in all cases it was found that the slope of the hydraulic gradient was steeper than 1 in 4, but with one exception flatter than 1 in 3. The American investigations carried out in a number of dams from 50 to 90 feet in height showed that the dams were completely saturated as far as the puddle core wall, and that moisture passed through the puddle wall with a loss of head of about 10 feet; that the line of saturation downstream of the puddle wall varied with the material. When watertight, the slope was 35 per cent or about 3 to 1, and where very porous, 10 to 1.

The Board deduced from their observations that with carefully selected material a drop of 17 per cent of the head in the reservoir may be expected at the puddle wall, and a slope of saturation of 23 per cent, or 5 to 1, in the rear of the dam.

They finally reported as follows:—

"On this basis, which is a liberal one, the maximum height to which an earth embankment, with its top width 20 feet above water line, and with outside slopes of 2 to 1, can be built with safety is 70 feet."

On the basis of Indian practice Hill recommends that the section of a dam should be such that a line drawn from the point where the full supply level cuts the upstream slope of the bank to the outer toe should not be less than 4 to 1.

For South African conditions it is recommended that at least one-third of the section of a dam on the lower side be composed of po ous material grading through from second-class earth to rock-fill; that a substantial toe of hand-packed rock-fill be placed at the foot of the downstream slope, and that the gradient from the inner edge of this toe to the intersection of the upstream slope with the full supply level be not steeper than 1 in 3 in the case of Karroo soils, or steeper than 1 in 4 in the case of black clay soils. The above remarks apply to earth dams of from, say, 40 to 100 feet in height, the stone toe in the case of high dams being from 20 to 40 feet in width.

The drainage in the foundations and lower half of the dam must be perfect, and it is advisable to introduce a system of French drains at natural surface level, which discharge by means of a main drain of this type clear of the toe.

A French drain should likewise be constructed along the foot of the stone toe and on the lower side thereof.

Small dams of less than 40 feet in height need not be built to such an ample section. The stone toe may be omitted, but the assumed hydraulic gradient to the outer toe should never be steeper than 1 in 3.

The "free board" of the dam or height of the crest above full supply level depends largely upon the high flood level allowed. This should be kept as low as possible, compatible with economical spillway design. The height of the crest above the high flood level should never be less than 3 feet. The height actually adopted will depend largely upon the "fetch" of the waves and on the height of the dam. In the river sections and other high portions it is advisable to make the free board allowance about one foot more than in the lower portions of the dam.

The top width of the dam varies with the height, and should be to some degree in proportion to the general scale of the dam. The question of maintenance should also be considered. In the case of small dams below 30 feet in height, the width may be 6 feet, increasing to, say, 15 feet in the case of a dam 75 feet high. When it is necessary to carry a public road along the crest of the dam, the width may have to be increased to 25 feet or more. To achieve this it is not necessary to increase the base width of the dam to any material extent. The required width can be obtained by steepening the side slopes of the dam above full supply level. Where a road is carried, it is advisable to build a low wall along the edge nearest the reservoir, both for purposes of ordinary public safety and to protect the road from spray during periods of high wind.

Side Slopes.—In the case of dams of low height, the slope of the water face may be 2 to 1; for dams between 25 and 50 feet high $2\frac{1}{2}$ to 1; and for dams over 50 feet high 3 to 1. The down-stream slope may vary from $1\frac{1}{2}$ to 1 in the case of low dams, to $2\frac{1}{2}$ to 1 in the case of high dams, the correct slope being determined by the limiting permissible hydraulic gradient, as previously indicated.

For the widening of the crest required to carry a road the top section of the dam may have slopes as steep as $1\frac{1}{2}$ to 1. There are certain drawbacks to this steepening of the side slopes, and where possible the slopes should be maintained at a uniform angle.

Where it is desired to increase the base width owing to an anticipated flat hydraulic gradient material can be saved by working with a downstream slope of, say, 2 to 1, and introducing one or two berms of 8 or 10 feet in width; but in such cases surface drainage must be provided for, otherwise damage is likely to occur by erosion. The attached sketch (Fig. 16), shows an example of a section to be adopted in the case of a high earth dam. This section embodies most of the recommendations made above.

Allowance for Settlement.—No matter how well the earthwork of a dam is consolidated during construction settlement will occur as the height increases and the earthwork is exposed to the influence of rain, water in the reservoir, and the great weight of the mass of the structure.

It should be assumed that the vertical settlement will amount to from one-third to one-half inch per foot height, and construction profiles should be erected accordingly.

Casing of the Dam.—The casing of a dam on the water face and crest should not be considered as part of the structure when designing its section. The water face should be protected by a covering of stone pitching or concrete slabs, which will be proof against wave action and the burrowings of crabs, etc.

The protection of the water slope is of great importance, and economy in this direction may be dearly paid for. Where broken rock is plentiful, it makes the best covering if carefully and tightly packed. The stone pitching should vary from nine to eighteen inches in depth and should be bedded on a layer of at least four inches of gravel or spawls. Pitching should never be laid directly on the earth slope, as erosion and consequent failure of the pitching is then inevitable.

Too much stress should not be laid on procuring a neat and smooth surface of the pitching. A rough surface acts as a much more effective wave break than a smooth one.

Where pitching stone cannot be readily procured the surface may be protected by means of concrete slabs laid *in situ* on a bed of gravel or ballast.

Cases occasionally arise where the depth of sand and silt below the bed of the dam is so great that the carrying down of a cut-off wall to bed-rock is economically impossible. If the dam to be built is of small height watertightness may be assured by greatly increasing the base width.

Thus the Brand Vlei dam near Worcester, South Africa,

FIG. 15.—Example of heavy stone pitching showing gravel layer underneath.

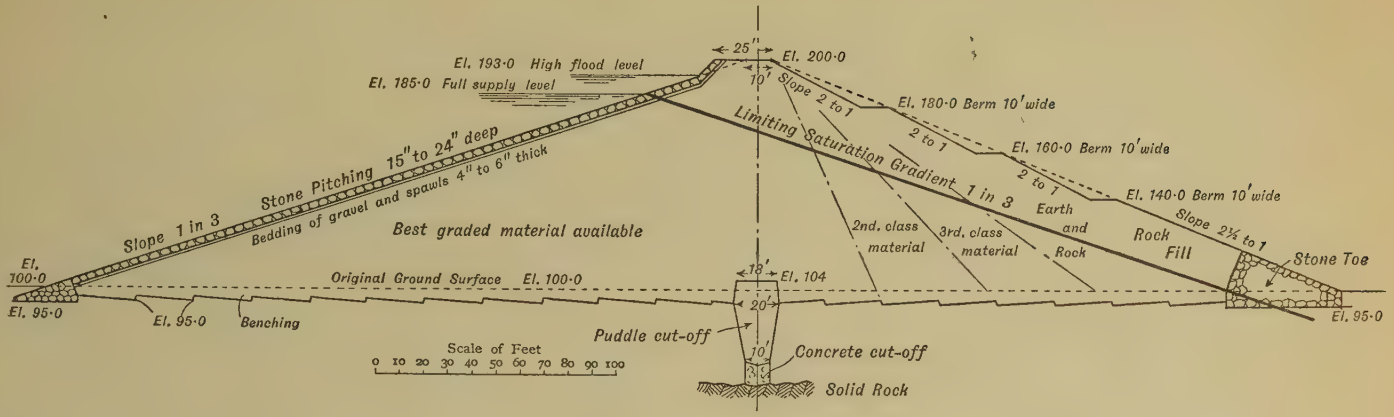


FIG. 16.—Typical section of high earth dam.

is bedded on a very thin and broken layer of sandy loam. Below this layer there is about 80 feet depth of quicksand, gravel, and other loose material. The dam is only 20 feet in height, and nearly 3,000 feet long, and impounds 35,000 acre feet at a depth of 15 feet. The base of the dam is, however, 138 feet in width. The upstream slopes range from 1 in 6 to 1 in 2, becoming steeper towards the top. There is also an apron of clay puddle 100 feet wide along the upstream toe. The limiting hydraulic gradient allowed for in this case is nearly 1 in 16 at full supply level, which is well within the limits of safety, as proved by experience gained with the great weirs of Northern India, which are mostly founded on fine water-bearing sand.

Consolidation of Earth Dams.—In South Africa dam construction often progresses during lengthy periods of drought, and as the usual and generally the best and cheapest mode of construction is by means of dam scrapers the material tends to become excessively dry and pulverized. Under these conditions the material will not bind properly, and failures have resulted from this cause. When working with scrapers, these should be drawn on to the dam on ramps running up the slope at a small angle with the long axis. The scrapers should travel as much as possible parallel to the dam and never at right angles to it.

The earthwork should be kept invariably higher at the sides than in the centre, so that the top surface will always be dished or hollow. The scrapers should travel a little distance along the dam before descending by one of the return ramps.

The work should be brought up as uniformly as possible over the entire length of the dam, and in layers not exceeding six inches in depth. On no account should the work be allowed to proceed in detached bits. When it is necessary to leave a gap, say in the river bed for the final closure, the sides of the gap must be stepped down, the treads of the steps being broad in comparison with the "risers." After each layer has been properly spread and all clods broken it should be moistened, if dry, by sprinkling a sufficient quantity of water on it to make it pack or bind. Rolling and moistening should proceed simultaneously.

Where in the upstream half of the dam it is necessary to mix several soils to obtain the best material, scraper loads should be brought on to the work in the proper proportion and mixing should be effected whilst spreading the earth.

No borrow pits must be excavated close to the toes of the

dam. In the case of small dams less than twenty feet in height, no borrow pit should be nearer than the final height of the dam. In the case of higher dams, borrow pits should be removed to a distance equal to one and a half to two times the height of the dam.

Similarly the allowable depth of borrow pits should in no case exceed six inches for a distance from the toe equal to the height of the dam. Thus at a distance of four times the height of the dam from the toe the depth should not exceed two feet.

Other details concerning the construction of earth dams may be found in several standard books on the subject, more especially in "Indian Storage Reservoirs," by W. L. Strange.

HYDRAULIC FILL DAMS

Earth dams built by the hydraulic fill method have in recent years become popular in the United States of America.

The process consists in transporting, sorting, and depositing earth and more particularly fine clay by flowing water. Works of this class have received a great deal of detailed attention in technical papers and in the technical press, frequently in connexion with failures during the construction period due to sloughing.

This type of dam is unsuited to South Africa, as the essential conditions for hydraulic sluicing and transporting of the material to be deposited in the dam do not exist. This method of dam building therefore will not be described.

SOLID MASONRY OR CONCRETE DAMS OF GRAVITY SECTION

It is unnecessary to treat in detail the design and construction of dams of this class, as the subject has been exhaustively dealt with in a large number of excellent textbooks, and, speaking generally, requirements as regards foundations and methods of construction are the same in South Africa as elsewhere.

This type is the most permanent and is generally considered the safest, and requires the least amount of maintenance.

This type of dam has the further advantage that it can be designed as a weir, so that surplus water can be discharged over the crest. When the dam is not of great height it is a common practice to use the whole or a portion of the crest as the waste weir, thus dispensing with the necessity of a separate spillway. There are numerous examples of such solid spillway dams in

South Africa, but they are mainly confined to town water supply schemes: for example, the Bulk and Sand River dams of the Port Elizabeth Waterworks, which are seventy-six feet and fifty feet high respectively.

The construction of gravity dams is generally simple and straightforward and allows plenty of scope for the use of large plant, which materially diminishes the unit cost of the concrete.

As the main factor in a dam of this class is its weight, it is possible to economize greatly in the use of cement. The essential requirements are that the upstream face and base of the dam should be watertight, that the skin generally should be composed of materials which will resist weathering and protect the hearting from any possible weathering action of the atmosphere or percolating water, and that the material comprising the dam as a whole shall be capable of withstanding the maximum crushing and shearing stresses with a considerable margin of safety.

To comply with these conditions a rich and dense concrete made of the best materials is essential on the faces and in the base of the dam, but the greater mass of the structure may be built with a weaker mixture, and with materials complying with a much less exacting specification than would be necessary for concrete in the base and exposed parts of the dam.

In the case of a very large dam of this type recently constructed in South Africa, the Lake Mentz dam, the rock immediately available at the site of the dam, viz. Dwyka conglomerate, though very hard and of high specific gravity, was not considered to be suitable for face work owing to an occasional tendency to disintegrate slightly on exposure to the atmosphere. Really good natural sand for concrete was not available locally. The nearest source of good durable rock (quartzite) was some distance away, and sand manufactured by crushing this quartzite rock, though of good quality, was very costly and difficult to produce in large quantities. It was therefore decided to construct the dam with a skin, base, and top of 6 : 3 : 1 concrete composed of quartzite rock and crushed quartzite sand, and a hearting of much weaker concrete, viz., 10 : 5 : 1 composed of Dwyka rock and washed river sand. By adopting this method of construction a very large saving of money was effected without in any way prejudicing the efficiency of the structure.

When adopting weak concrete mixtures of this kind there should be a liberal allowance of plums, or displacers, in order to

raise the specific gravity of the mass to as high a figure as possible. The saving of concrete in this case, by the use of "plums," is a minor consideration, as in many cases the high cost of handling the plums counterbalances any saving in concrete. In the case of some large dams in South Africa, owing to the unsuitability of the rock for the purpose, the placing of plums actually enhanced the cost of the structure.

In order to minimize uplift stresses dams of this type should be provided with an efficient system of internal drainage.

In the case of the Lake Mentz and Kamanassie dams, for the design of which the author was responsible, vertical weeper drains, six inches diameter, are provided a few feet back from the upstream face. These drains are spaced ten feet apart, from the foundations upwards into a drainage gallery traversing nearly the whole length of the dam, and from the gallery upwards to near the full supply level. The drainage gallery communicates with a main outfall drain at the highest point of the dam, the effluent discharging through the dam into the river channel.

In the case of the Kamanassie dam foundation drains have been provided also near the toe of the dam.

With a complete system of internal drainage of this kind it is unnecessary to add materially to the section of the dam to compensate for uplift pressure. In the examples cited above full allowance was made for uplift pressure at the base, but this allowance was gradually decreased towards the top of the dams.

The provision of both a complete internal drainage system and the full extra allowance of cross section to compensate for uplift appears, in the author's opinion, to be unnecessary, especially if special care is taken to provide for as impervious a water face as possible.

As all monolithic masonry or concrete structures of considerable length will crack, owing to shrinkage due to large variations in temperature, it is very necessary to provide artificial joints at certain intervals to obviate this tendency.

If such joints are not provided cracks may form on lines of weakness in such a manner as to impair the stability of the structures.

Temperature cracks, even if they occur in vertical planes at right angles to the axis of the dam, and therefore in no way dangerous to the stability of the dam, are, to say the least of it, unsightly.

The best form of contraction joint has still to be invented.

For the present the practice in South Africa is to provide these joints at intervals of from fifty to one hundred feet. Each contraction joint unit of the dam should be built separately with smooth faces, so that no adhesion between adjoining faces can take place.

Leakage through the joints is prevented by means of copper sheeting eighteen to twenty-four inches wide, and one-eighth to one-quarter of an inch thick, covered on either side with two-ply bituminous sheeting. The copper strip, which is continuous

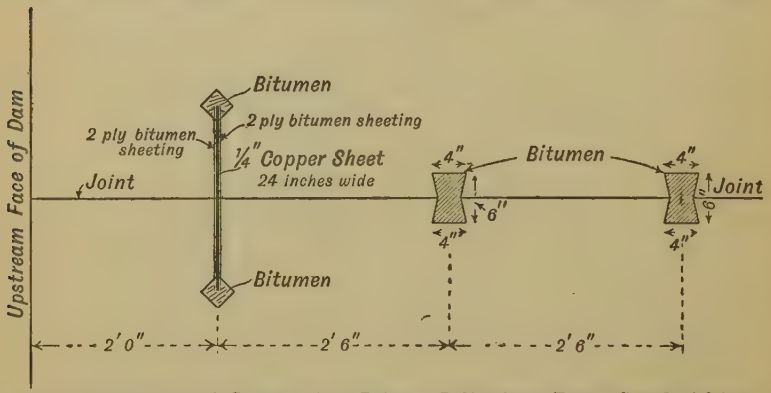


FIG. 17.—Details of Contraction Joints, Calitzdorp Dam, South Africa.

from top to bottom, together with the bituminous sheet covering is held in wedge-shaped grooves or dowels in the opposing faces of the contraction units of the dam, the space between the copper strip and the side of the groove or dowel being filled with bitumen.

In the case of the Calitzdorp dam in the Oudtshoorn district of the Cape province two additional checks were introduced consisting of opposing wedge-shaped key-ways filled with bitumen. It is doubtful whether the latter are really effective unless means are provided for removing the bituminous filling from time to time. Illustrations of such contraction joints are shown in Fig. 17.

Under the climatic conditions prevailing over a large part of South Africa experience shows that contraction joints in large dams should be provided at intervals of not greater than fifty or sixty feet.

As a typical recent example of gravity section concrete dam particulars of the Kamanassie dam in the Oudtshoorn district

of the Cape province are shown in Fig. 18. This dam is constructed with 1 : 4 : 8 concrete throughout. At the commencement it was to be cased with 1 : 3 : 6 concrete, but this was abandoned later when the excellence of the weaker mixture had been demonstrated. The ballast consists of crushed Bokkeveld

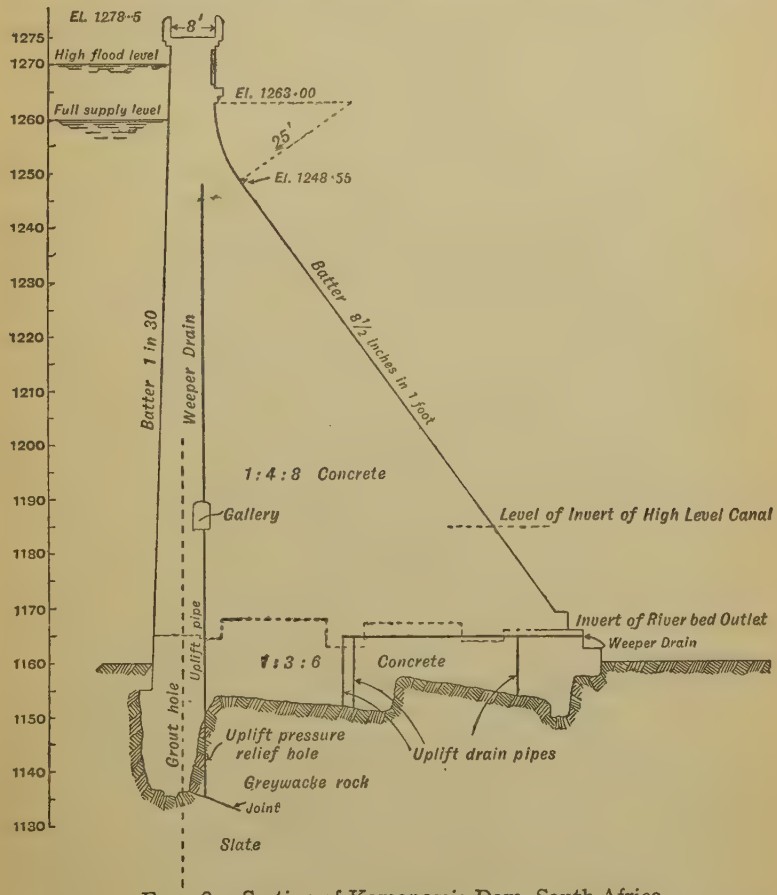


FIG. 18.—Section of Kamanassie Dam, South Africa.

Greywacke rock. This is a hard rock with a slaty cleavage and does not crush very well. No attempt was made to crush small, and stone crusher run was used, which gives everything from dust to pieces six and eight inches long in one direction. The long pieces are generally spawls about two inches thick.

The sand is quartzitic and very good. As the result of a large number of tests with six-inch cubes the crushing strength of the 1 : 4 : 8 concrete was found to be very uniform and averaged 1,736 lbs. per square inch at six months, and was practically equal in strength to the 1 : 3 : 6 mixture.

The chief drawback to gravity type of dam, as compared with other types, is its high cost, but in many cases it provides the only practicable alternative, especially when both the length and height of the dam are considerable.

ARCHED MASONRY DAMS

Next to solid gravity dams the most efficient types of structures are represented by arched dams, the stability of which depends mainly upon their action as an arch, the water pressure being transmitted to the abutments. This type can only be built if the site is narrow and the foundations good and the sides composed of sound hard rock which is capable of taking the thrust. In many respects it is an ideal type, as, while being a permanent structure, it contains very much less material than the gravity dam. Unfortunately sites suitable for this type are comparatively rare.

A combination of arch dam with gravity section tangents forming abutments is beginning to find favour and may possibly provide a solution of many difficulties.

There are many well-known examples of arch dams in the United States of America and in New South Wales. In South Africa the only notable example of this type is the Hartebeest Poort dam on the Crocodile River in the district of Pretoria in the Transvaal province.

This dam is built in a narrow gorge or poort traversing the Magaliesberg range, which is composed of quartzite rock. The dam is of the constant angle arch type, and is 161 feet high above river bed, and 193 feet high above the lowest foundation. It is built throughout with 1 : 3 : 6 concrete. The Hartebeest Poort dam is an example of a type likely to increase in favour, viz., of a constant angle arch with abutments of gravity section.

There are several points of interest in connexion with arched dams which it is advisable to enlarge upon, as it is desirable to familiarize engineers with this type and its near relation the multiple arch dam, which will be more specifically dealt with later on.

If a gravity section masonry or concrete dam is given a curve convex upstream the factor of safety against sliding or shearing is increased, as it can neither slide not overturn without crushing either the masonry in the dam itself or its abutments. Powers of resistance to failure other than mere weight are then brought into play. If the radius of the arch is reduced the stresses of the structure, considered as a gravity dam, are likewise reduced, and it becomes possible to effect a considerable reduction in the thickness of the dam.

Captain Garrett, in Government of India Technical Paper No. 170, 1904, proved that the most economical arch dam is one which subtends an angle of 133 degrees 34 feet. This however assumes the thickness of the arch at the water surface to be zero. In practice an arch subtending an angle of 120 degrees will give very nearly the most economical structure. This important principle was also developed by Garrett in a paper read by him before the Simla (India) Engineering Conference in 1913. It is generally attributed to Jorgensen, who dealt with the matter in his paper on the Constant Angle Arch Dam (Trans. Am. Soc. C.E., Vol. 78, page 685). Whilst the credit is due to Garrett, the latter paper is more accessible. It is obvious therefore that for economical construction the width of the gorge between abutments is limited. Where the width exceeds this limit by a comparatively small amount it is sometimes possible to combine the gravity and arch types, the gravity section on one or both flanks forming abutments for the arch section.

Where this type of structure can be economically employed it is greatly superior to the gravity dam, being very much less costly to build and at the same time equally permanent. It is unfortunate that suitable sites are somewhat rare.

The fact that an arch slice in a dam subtending an angle of, say, 120 degrees is the most economical and subjected to minimum stresses is due to the fact that the arch when loaded is deflected proportionally to the square of the upstream radius, and in what is known as the " constant angle " type this radius may be much shorter near the base where the gorge is narrow than at higher levels. Therefore for a given unit stress the deflection at the base will be proportionately less than the deflection required at the top.

The theory of the design of constant angle arch dams has been very fully treated in various textbooks and technical papers,

most of which are readily accessible, and will not be given here. Special attention is invited to the following publications :—

Creager, W. P., " Masonry Dams." John Wiley & Sons, New York.

Morrison, C. G., and Brodie, O. C., " Masonry Dam Design." John Wiley & Sons, New York.

Garrett, A. ff., " A Proposed New Method of Design for Masonry Dams." Govt. of India Technical Papers, No. 170.

Jorgensen, L. R., " The Constant Angle Arch Dam." Trans. Am. Soc. C.E., Vol. 78, p. 685 (1915).

Jorgensen, L. R., " Arch Action in Arch Dams." Proceedings Am. Soc. C.E., Vol. 83, p. 316 (1919-20).

This particular type of arch dam should be adopted in preference to other arch forms, whether in the case of single or multiple arch dams.

The arch stress at any height can be calculated from the formula

$$p = \frac{qr_u}{t},$$

where p = the stress in the arch at any elevation.
q = unit load taken by the arch at any elevation.
r_u = radius of the upstream face at any elevation.
t = thickness of the arch at any elevation.

This formula presumes that the deflection of the arch at the base is unrestrained. q/t and r_u are presumed to be constant.

This formula is only strictly correct if the arch radius is large compared with the arch thickness.

The value of q must be selected with care. Examples are in existence, viz., the Upper Otay and Bear Valley dam, where the stress amounts to no less than 120,000 lbs., or 54 tons, per square foot. In most cases the allowable stress varies from 10 to 20 tons per square foot.

For South African conditions moderate limiting stresses are advisable, and should not exceed 15 tons per square foot, according to the nature of the material available for concrete.

In the case of the three highest arch dams so far constructed —viz., the Shoshone dam, which has a total height of 305 feet ; the Salmon River dam, 220 feet high ; and the Pathfinder dam, 210 feet high—the maximum arch stresses vary between 11 and 13 tons per square foot.

In view of the uncertainty in the determination of the

induced stresses in the dam due to normal loading in combination with temperature and other indeterminate stresses a high factor of safety should be employed. Creager recommends that the maximum stress calculated according to the above formula should not exceed one-eighth to one-twelfth of the ultimate strength of the masonry or concrete. In view of the uncertainty regarding these temperature and other indeterminate stresses, it is also advisable to neglect entirely the effect of beam action in the arch.

Arch dams can be constructed as spillway dams, but in such cases the profile of the dam should be made such that the sheet of water falling over the crest will not leave the surface of the dam, and for this reason it is not possible to design the dam on the constant angle principle, as this generally gives an almost vertical downstream face.

Some very interesting suggestions concerning arch dams are contained in a paper on "Arch Action in Arch Dams," by L. R. Jorgensen (Paper No. 1431, Am. Soc. C.E., Vol. LXXXIII, 1919-1920).

Arch dams, and mainly those of heavy sectional area, do not develop arch action before considerable load has been applied.

At present comparatively low working stresses are used in ordinary dam practice, which produce safe structures, though formulæ used in the design do not consider everything.

If arch action could be developed from the very beginning higher unit stresses could be used with safety. The reasons for the delay in the arch action coming into play are :

(1) Shortening of arch rib due to shrinkage ;
(2) Because arch is fastened to foundation.

Shortening of arch rib is due to the natural shrinkage of the concrete as it sets, the dissipation of chemical heat, the permanent set after reservoir has once filled, and the fact that dams are often built during season of highest temperature.

Shortening of arch rib is plainly visible in dams provided with contraction joints. These will open as much as one-quarter and three-eighths of an inch if joints are spaced sixty feet apart.

In cases where contraction joints have opened up the water level will have to rise a certain height in order to throw sufficient load on the structure to force the dam, acting as a cantilever, to deflect in a downstream direction, before the voussoirs, i.e. bodies between the contraction joints, come into contact with

one another. Only from that moment does the remainder of the water load divide up between the arch, the curved beam, and the cantilever.

Owing to the dam being fastened to the foundations arch action is largely prevented near the foundation. The load on the lower portion of the arch causes shearing stresses in the concrete next to the rock and for some distance above and below it. These shearing stresses cause an elastic deformation of both dam and rock foundation to take place in a downstream direction, especially at the crown of the arch, where the movement in that direction is greatest.

The paper gives deflection curves for the Salmon Creek dam, near Juneau, Alaska. This is a constant angle arch type dam 168 feet high with a crest span of 550 feet. Maximum deflection at crest with reservoir full:

Winter 1·0 inch. Summer 0·4 inch.

The deflection curves show clearly effect of portions of the dam built in coldest months.

Jorgensen describes a method of grouting the spaces between adjacent walls of the contraction joints, which puts initial compression into the arch whereby the shearing and cantilever stresses can be cut down to perhaps half the value they would otherwise have been. Grouting must be done when joints are at their maximum opening.

HOLLOW DAMS

Hollow dams are a type of comparatively recent growth, and have developed more particularly with the introduction of reinforced concrete construction.

There are two main types. The "Ambursen" buttress type (which is a reinforced concrete type) is a development of the timber dams which were common in the pioneering days of the western States of America. Their concrete successors consist of a series of piers or buttresses supporting a sloping deck of reinforced concrete slabs.

The other type is a development of the concrete arch dam, and consists of a series of plain or reinforced concrete arches between the piers or buttresses.

An important advantage of these hollow dams is that the pressure on the foundations can be greatly reduced by spreading the base and distributing the pressure equally over it, but their

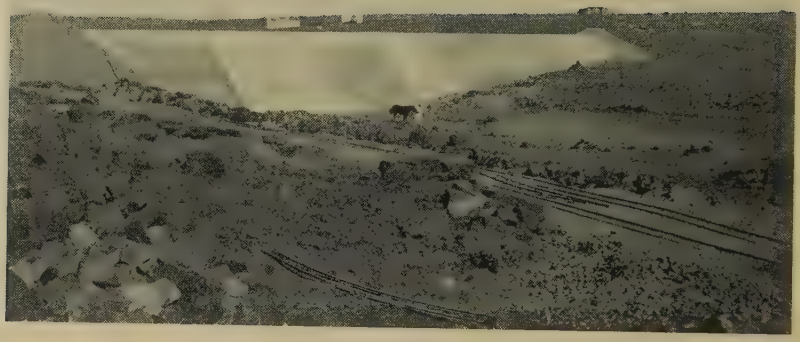


FIG. 19.—Reinforced concrete dam—Ambursen type—from above.

FIG. 20.—Same dam as Fig. 19, from below.

height is limited by the permissible stresses in the piers and decking. The upstream decking can be made at any angle, but in most cases a slope of about forty-five degrees is adopted. On the downstream side the buttresses are either vertical or have a small batter.

"Ambursen" dams are frequently designed as spillway dams, in which case the crest must be made to fit the shape of the sheet of water flowing over the dam. The prevention of percolation under a hollow dam is as essential as in the case of any other type, and this is attained by providing a substantial cut-off wall along the upstream edge, to which the decking must be securely attached.

The spacing of the buttresses is limited by the economical span of the deck and by the allowable pressure in the buttresses. In the case of "Ambursen" dams, spans of from fifteen to twenty-five feet are common. In the case of multiple arch dams they are greater and range from thirty to fifty feet. The calculations for designing "Ambursen" dams will be found in many modern textbooks and will not be dealt with here.

Their adoption in South Africa is at present very limited, owing to the very high cost of reinforced concrete construction. The only notable example of a dam of this type is a subsidiary dam of the large storage works of the Smartt Syndicate, near Britstown, of which some views are given in Figs. 19, 20, 21.*

The Multiple Arch dam was developed by Captain Garrett, R.E., in India, and is often known as the Garrett type. Its development followed on the recognition of the obvious advantages of the single arch concrete dam, which, however, is limited in its application to comparatively narrow sites.

The type did not, however, originate with him, as the first example of its kind, viz., the Meer Alum dam in India, was constructed about the year 1800. This dam is about half a mile long, and consists of twenty-one arches ranging from 70 feet to 147 feet in span.

One of the main advantages of the multiple arch type, as compared with the flat deck or Ambursen type, is that it can be built in ordinary mass concrete, without reinforcement. Apart from securing greater permanency owing to the absence of steel reinforcement, which might corrode, it makes the structure a very much cheaper one to build. There will certainly be more concrete in an unreinforced concrete multiple arch dam than in

* See also Fig. 21, facing page 202.

the same type built in reinforced concrete or in the Ambursen type, but this disadvantage is greatly outweighed by the saving in steel reinforcement, skilled labour, and simpler form work.

In order to ensure against failure of the entire work owing to the possible failure of one arch it is advisable to make the arches subtend an angle of 120 degrees or more, so as to make each buttress independent of its neighbours in taking the thrust of the arches abutting on to it.

The flatter the arches are made the more solidly should they be built so as to obviate any possibility of failure.

As the 120 degrees arch is more economical as regards forms and labour, and presents, moreover, a smaller surface exposed to the water, this arch would appear to be more suitable.

A multiplicity of short spans and thin buttresses would theoretically require less concrete than larger spans and thicker buttresses, but experience seems to indicate that spans of from forty to fifty feet, according to height, are the most practicable and economical.

The arches are not subject to uniform loading, especially in the higher portion of the dam, and they should be designed accordingly.

As previously stated, an arch subtending an angle of 120 degrees will give very nearly the most economical construction.

Amongst the advantages of multiple arch dams, as compared with gravity dams, are the following: Saving in first cost; greater adaptability to local conditions; greater safety against upward pressure under the foundations. The arch ring is comparatively so thin that the pressure of any water percolating through the foundations would certainly be relieved by escape downstream of the arch ring. For the same reason no dangerous upward pressure could be carried under the piers.

Arch dams are more adaptable to changes caused by temperature and shrinkage due to initial setting.

There are distinct limits, however, to the height to which multiple arch dams can be built, owing to the high stresses developed, and Captain Garrett places the limit of height at from 100 to 120 feet.

Other modifications of the multiple arch type have been proposed, notably the Dillman type (Am. Soc. of C.E., 1902) and the Ryves parallel slice dam. The former is good theoretically, but more costly than the Garrett type. The latter claims to be safer than the Garrett type, but this appears to be doubtful.

When designing multiple arch dams, these should rest upon a solid concrete foundation or cut-off wall, and massive concrete abutments on both flanks. Whilst at least one example of a multiple arch spillway dam exists, viz., the Orland dam in California, the type is not particularly suited for the purpose.

An excellent description of two modern examples of this type, viz., the Gem Lake dam 700 feet long with a maximum height of 112 feet, and the Agnew Lake dam 280 feet long and 30 feet high, together with an analysis of the stresses in multiple arch dams, is contained in a paper by L. R. Jorgensen read before the Am. Soc. C.E. (*vide* Paper No. 1396, Vol. LXXXI, 1917).

Before leaving the subject of concrete dams a few remarks on the subject of materials available for construction may be of interest.

Wherever the geological formation consists of rocks of pre-Karroo age little difficulty is experienced in obtaining really first-class materials, viz., rock for concrete ballast and natural sand, the latter being mainly quartzitic. In the vast area covered by rocks of the Karroo series, comprising the greater part of the Cape and Free State provinces, the position is much less satisfactory. The Karroo rocks comprise sandstones and shales, the latter predominating, and these are soft and disintegrate on exposure to the atmosphere. At the base of the Karroo beds, and outcropping over a comparatively narrow width all round the outer edge of the great Karroo basin, are to be found the Dwyka beds, the most characteristic rock of which is the Dwyka conglomerate. This rock is of glacial origin, and consists of a hard, fine-grained shale with a slaty cleavage and containing boulders of various rocks, mainly granite and quartzite. This rock is hard and dense, but is inclined to disintegrate on exposure to the atmosphere. If used for concrete in dam building it should be encased in concrete made with rock of a more durable nature.

Hard sandstones are to be found occasionally, but seldom close to sites for reservoir dams. The shales, unless indurated by contact with igneous dykes, are useless for permanent engineering structures. The greater part of the Karroo area is intersected or covered by dykes and sheets of dolerite. This is a very hard igneous rock, and, when fresh, suitable for concrete or masonry work.

Dolerite, Dwyka conglomerate, and Bokkeveld slates crush badly and yield a large percentage of elongated pieces, flat pieces, and splinters. The hard sandstone crushes much better.

Karroo sands are nearly always inferior. Pure dolerite sand is good, but hardly ever available in large quantities. Generally speaking, the sand in Karroo rivers is largely composed of shale particles, which are rounded and soft, and contain a large percentage of mud and other objectionable matter. Whenever it becomes necessary to use these river sands thorough washing is essential. In some instances it has been necessary to make sand for concrete artificially by crushing and rolling quartzitic rock.

The Free State presents the greatest difficulties, both in respect of rock and sand. The dolerite sheets and dykes are badly decomposed, and sometimes the only rock available is a very limited amount of sandstone.

Throughout the folded belt all around the coast, the rocks consist of Table Mountain sandstones and quartzites, Bokkeveld greywackes and Witteberg quartzites and both the rock and the sands derived therefrom provide excellent material for concrete.

In the annexed Table No. XXV are given some mean crushing strengths of concretes tried or used in the construction of some recent large dams in the Cape and Transvaal provinces.

ROCK-FILL DAMS

When describing earth dams it was stated that these should have fine and impervious material on the water side, and grade gradually to coarse material and rock-fill on the downstream side. The so-called rock-fill type is merely a further development of this idea. In this case the main mass of the dam is composed of loose rock, which provides a massive heavy buttress to take the thrust of the water pressure, with a cut-off wall and solid decking on the water face to make the structure watertight. The earliest examples of this class were covered with carefully caulked wooden decking, the surface being protected with asphalt or similar material. Later, central core walls of masonry, concrete, or steel were introduced. The obvious and most efficient design, however, is to place a permanent decking on the upstream face, so that the entire mass of rock-fill is employed to take the water pressure.

A very substantial cut-off wall along the upstream toe is essential, and care must be taken to ensure that all percolation likely to be harmful, i.e. such as would be at all prejudicial to a concrete or earth dam, is entirely prevented.

TABLE XXV

Mean Crushing Strengths of Various Concrete Mixtures used in the Construction of Some Recent Dams in South Africa.

Rock Used in Concrete	Sand Used	Concrete Proportions			Age of Test Blocks	Size of Test Blocks	Crushing Stress in lbs. per sq. ft.
		Ballast	Sand	Cement			
(1) Dwyka conglomerate	Washed river sand	10	5	1	6 months	6″×6″×6″	165,222
(2) Do.	Shale sandstone and dolerite	10	5	1	12 do.	6″×6″×6″	81,000
(3) Witteberg quartzite	Crushed quartzite	6	3	1	2 do.	6″×6″×6″	214,335
(4) Do.	Do.	6	3	1	12 do.	6″×6″×6″	294,964
(5) Do.	River sand—same as (1)	6	3	1	6 do.	6″×6″×6″	257,376
(6) Dwyka—same as (1)	River sand—same as (1)	6	3	1	12 do.	6″×6″×6″	102,144
(7) Greywacke (Bokkeveld)	River sand—quartzitic	6	3	1	3 do.	6″×6″×6″	203,660
Do.	Do. do.	6	3	1	6 do.	6″×6″×6″	244,900
Do.	Do. do.	8	4	1	3 do.	6″×6″×6″	188,150
Do.	Do. do.	8	4	1	6 do.	6″×6″×6″	250,000
Quartzite (Magaliesberg)	Do. do.	4	2	1	1 month	6″×6″×6″	436,150
Do.	Do. do.	6	3	1	1 do.	6″×6″×6″	391,955
Do.	Do. do.	6	3·5	1	1 do.	6″×6″×6″	326,965
Do.	Do. do.	7	3·5	1	1 do.	6″×6″×6″	335,357
Do.	Do. do.	7	3·5	1	2 months	6″×6″×6″	445,387
Do.	Do. do.	6	3·5	1	2 do.	6″×6″×6″	385,175
Do.	Do. do.	6	3	1	3 do.	Cylinder 8″ diam. 16″ high	206,812
Do.	Do. do.	4	2	1	1 month	Do. do.	179,327
Diabase	Do. do.	6	3	1	1 do.	6″×6″×6″	278,163
Dolerite	River sand—Dolerite, shale and sandstone	5	3	1	60 to 70 days	6″×6″×6″	96,595
Do.	Do. do.	7	3	1	60 to 70 days	6″×6″×6″	63,824

The decking should be slightly flexible, and be composed of reinforced slabs with efficient contraction joints. Should any slight settlement occur in the rock-fill the decking will adapt itself to the distortion and will at once give a warning of what is happening. This provides a further reason for placing the decking on the upstream face. A central core wall is hidden from view. It is delicate structure which may be cracked during the process of dumping the rock-fill, or it may crack subsequently as the result of settlement. In either case, if serious leakage occurs when the dam is complete, repairs would involve an enormous amount of labour.

The decking must be securely bonded into the top of this cut-off wall.

The floor of the dam should be as free from fine loose material as possible and suited to take the weight of the dam. If there is any doubt about the material under the base of the rock-fill it may be traversed by one or more additional cut-off walls parallel to the main cut-off wall, but of smaller section, and near the downstream toe a deep trench should be cut and filled with graded material which will act as a filter and effectively prevent any fine material from being carried away from below the dam by water finding its way under pressure below the dam.

It is equally necessary, of course, to secure a perfect cut-off between the decking and the rocky flanks of the valley.

In most cases a rock-fill dam is economically feasible only if the bulk of the fill can be quarried close to the flanks and above the top level of the dam and dumped from the site of the excavation directly on to the site of the dam. A long haulage of the rock-fill to the site would in most cases make the structure more costly than some alternative type of dam.

The rock-fill should be free from soil but should include all quarry waste.

The rock may be excavated economically by means of blasts at levels above the top of the dam without endangering the site-formation, provided that the strata are located so that effects of the explosions will not open up seams in the vicinity.

Much useful information concerning the construction of rock-fill dams is contained in N. M. O'Shaughnessy's paper on the "Morena Rock-fill Dam" in Vol. LXXV of the Am. Soc. of C.E., and in the discussion thereon. In this discussion Galloway calls attention to the necessity of having a substantial backing of derrick- and hand-laid rock on the water slope immediately

underneath the watertight skin. Failure to provide this packed layer is likely to cause settlement of the skin.

Although provision may be made for passing floods over rock-fill during construction, these dams should never be designed to act as spillway dams.

Experience with a notable South African example has shown that the structure can be made secure against failure due to passing floods over the top during construction by keeping the downstream edge of the fill horizontal and at a uniform height throughout the length of the dam, and maintaining this lip or sill a little higher than the rest of the structure. The downstream slope should be hand- or derrick-packed in the same manner as the upstream slope, but composed of much heavier stones, and in addition it should be covered and securely bound together by a heavy wire network having an eight to twelve inch mesh.

There are at present three examples of this type in South Africa, viz., Kriegers Poort (Hanover District), Kaffir River (Orange Free State), and Prins River. The last mentioned is the highest of the three, being 95 feet above the river bed, and 112 feet to the lowest foundation of the upstream cut-off wall. This dam is fully described in a paper by Mr. F. T. Patterson, M.Inst. C.E., read before the South African Society of Civil Engineers (Vol. XVI, 1918). Although the author was responsible for the design of this work he is of opinion that a more economical section might be adopted, and the Morena rock-fill dam constructed for the Southern California Mountains Water Company is an example of the most efficient design of such a structure.

In an article in the "Engineering News" of May 11, 1916, a correspondent gives a very interesting comparative table, showing, for a number of rock-fill dams actually constructed, the ratio of the applied load due to the water pressure, assuming a full reservoir, and the opposing force due to the weight of the rock-fill behind the watertight stratum. This is a somewhat rough-and-ready method of testing the relative stability of different dams, and we find that in the case of the rock-fill dams, which have failed, the ratios were approximately as follow:

Lower Otay Dam	1 : 2
Pecos River	1 : 2·35
Walnut Grove	1 : 2·6

Seven other rock-fill dams are mentioned with ratios varying from 1 : 2·1 to 1 : 5.

The Morena dam referred to above has a ratio of 1 : 5, whereas the Prins River dam has a ratio of 1 : 6·4. Some particulars of the design of the Prins River dam are shown in Figures 22, 23 and 24.

MOVABLE OR OPEN DAMS AND WEIRS

Under certain favourable conditions it is possible to store very large volumes of water under a low head in a river bed. Numerous examples of this kind are to be found in South Africa. The Vaal River is a case in point. The longitudinal section of this river shows a series of rapids linked together by long reaches of channel with very flat gradients, and a number of excellent storage reservoirs can be created by building low weirs on the crests of some of these rapids. One of these sites has been appropriated by the Rand Water Board, whose duty is to supply Johannesburg and the Witwatersrand generally with water.

This reservoir is created by a barrage of gates 25 feet high and contains, when full, 2,180 million cubic feet, the total length of the reservoir being 39 miles. Similar favourable sites exist at numerous other places along this river, some of which are certain to be exploited in the future.

In cases such as these it is however not feasible to construct a solid dam. In the first place, the afflux created during periods of high flood would seriously affect property abutting on to the reservoir and above it, and in the second place trouble with silt would be experienced in a comparatively short time. It may be said generally that in South Africa no permanent storage reservoir can be created wholly within the trough of a river by means of a solid dam or weir. In the few cases where this has been attempted the available storage capacity has been eliminated or very greatly reduced in a very short time, as, for example, in the case of the two dams in the Modder River belonging to the Bloemfontein Municipal Waterworks. Reservoir sites of this kind imply very low stream velocities, excepting during periods of high flood, when all rapids become drowned and a sufficient surface slope is created to deal with the discharge and its burden of silt. Blocking such a river during periods of flood makes it impossible for it to maintain its normal regime,

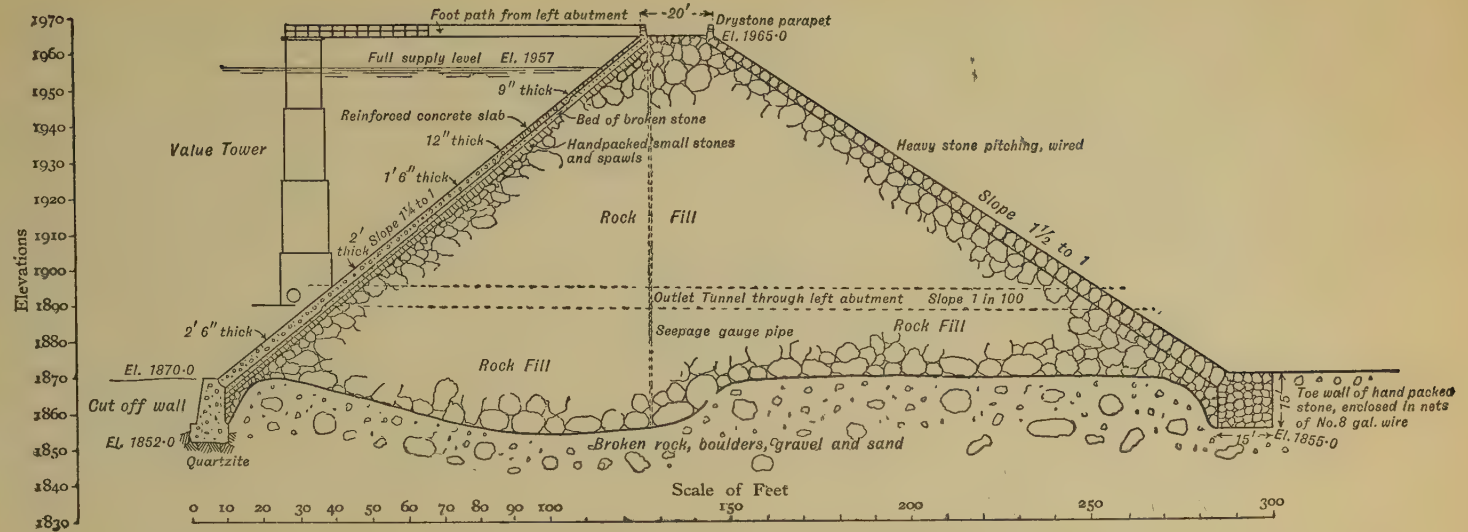


FIG. 22.—Prins River Dam, South Africa, cross section.

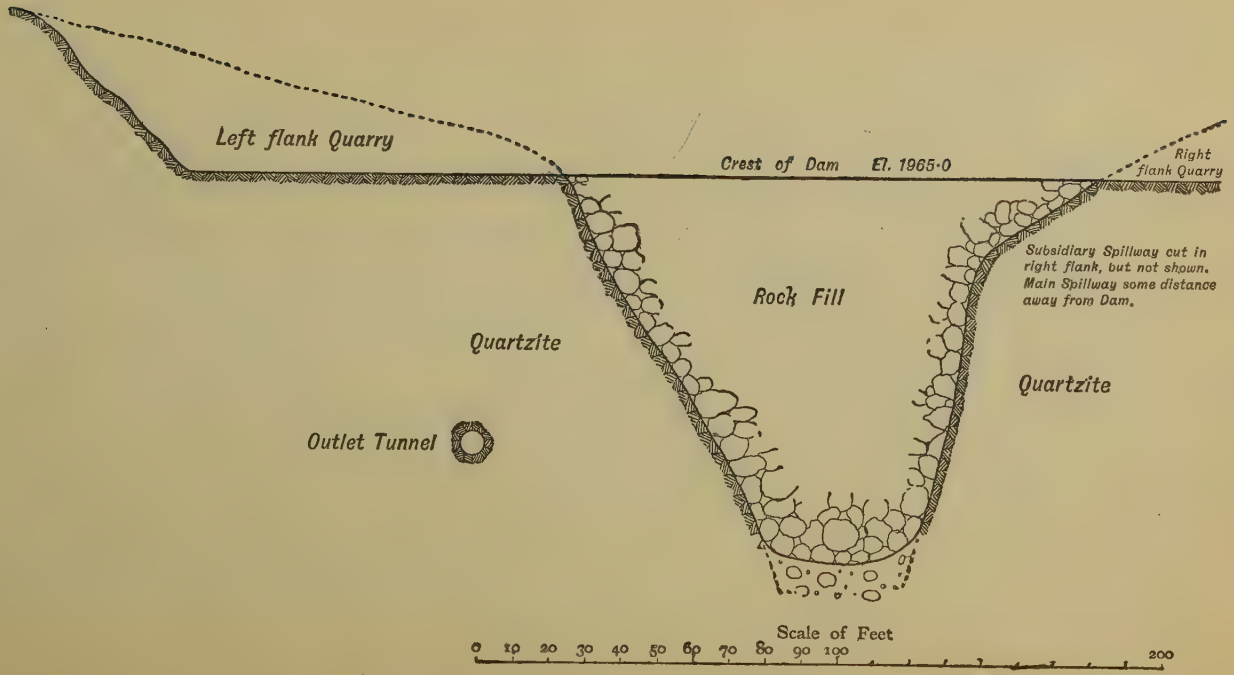

Left flank Quarry
Crest of Dam El. 1965·0
Right flank Quarry
Subsidiary Spillway cut in right flank, but not shown. Main Spillway some distance away from Dam.
Rock Fill
Quartzite
Quartzite
Outlet Tunnel
Scale of Feet
0 10 20 30 40 50 60 70 80 90 100 200

FIG. 23.—Prins River Dam, section along centre line.

and silting inevitably results. The only method of dealing satisfactorily is to effect the required heading up and storage by means of barrages or movable dams, which during periods of flood can be opened or removed in such a manner that the original regime of the river is, for the time being, restored. The reservoir can by this means be maintained entirely free from silt.

As the heading up required in nearly all cases for storage purposes is in excess of ten feet the use of drop shutters and

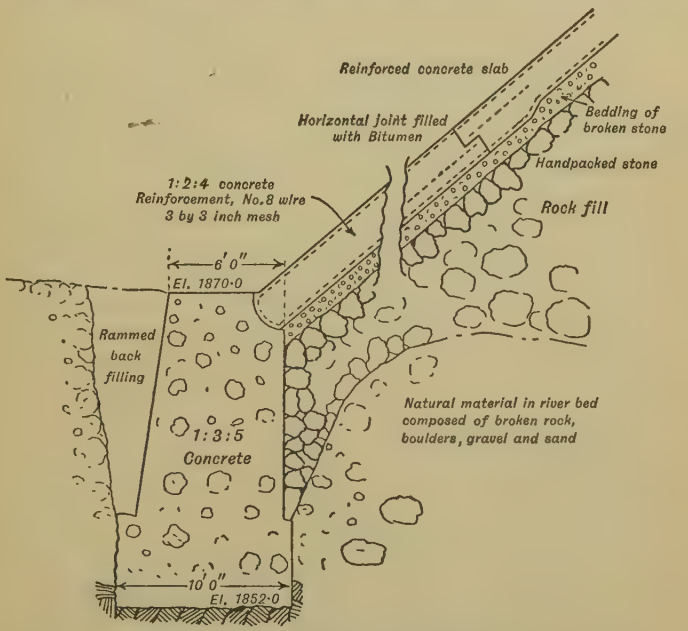

Reinforced concrete slab
Horizontal joint filled with Bitumen
Bedding of broken stone
Handpacked stone
1:2:4 concrete Reinforcement, No.8 wire 3 by 3 inch mesh
Rock fill
6' 0"
El. 1870·0
Rammed back filling
1:3:5 Concrete
Natural material in river bed composed of broken rock, boulders, gravel and sand
10' 0"
El. 1852·0

FIG. 24.—Prins River Dam, section of decking and cut-off wall.

similar devices is impracticable, and the only satisfactory method available is to design the barrage as a series of large gates working between piers so constructed that the gates can be speedily lifted clear of high flood level. South Africa has in this matter followed recent Indian practice, where structures of this kind have been employed for many years. These are in the form of series of large under-sluices, such as are associated with all the great canal-head works of Upper India, or in the form of great barrages, such as that at Balloki on the River Ravi, near Lahore in the Punjab. Such works are on the whole exceedingly simple, and consist of a masonry or concrete floor and sill, with

an apron extending some distance downstream, massive piers supporting the large steel gates, and a superstructure for carrying the machinery for operating the gates. Difficulties, where such exist, are nearly always confined to the foundations.

The sills of the gates may either be placed at the natural bed level or may be raised some height above it. The Rand Water Board barrage has its sill practically at bed level. In the case of the head works of the Olifants River Canal in the district of Van Rhynsdorp in the Cape province, which was the first work of this type to be built in South Africa, the sills of the gates are placed on the top of a solid masonry weir twenty feet in height, this raising of the sill being necessary in order to divert the water into the irrigation canal. In this case the storage behind the solid portion of the dam cannot be made use of and may with advantage be allowed to silt up.

In the earlier types of gate barrages, under-sluices, etc., the gates were comparatively small. In India they were generally twenty feet span, and three to six feet high, the required depth being obtained by using two or three tiers of gates. These gates were heavy and were hoisted directly on to the winches. Clearing a battery of under-sluices in anticipation of a flood was a tedious and laborious process, and a small army of Indian coolies was required for the purpose.

The introduction of "Stoney" sluices at a new canal-head works in Burma in the 'nineties opened the road to great improvements. Stoney sluices were fitted to some of the works in the Punjab shortly afterwards. In connexion with the great Triple Canal Irrigation Project in the Punjab the best form of sluice gates was made the subject of thorough investigation, and eventually a modification of the "Stoney" gate was designed and patented by John Ashford, Mechanical Engineer to the Punjab Irrigation Department, which has now become the standard type for India, and is manufactured in the local canal workshops. This gate is manufactured also by Messrs. Glenfield Kennedy, of Kilmarnock, Scotland. The author introduced this sluice gate into South African practice, and, after gaining experience therewith on the Olifants River, a standard pattern was designed, in collaboration with the manufacturers, which has been adopted for the large spillway sluices in the Lake Mentz dam, and was to be used on several other large works. Almost the identical design has been adopted by Mr. W. Ingham, M.Inst.C.E., Chief Engineer to the Rand Water Board, for the

FIG. 21.—Same dam as Fig. 19, from below, under construction.

FIG. 25.—Head works barrage of Olifants River Irrigation Canal, Van Rhynsdorp, Cape Province.

Fig. 26.—Vaal River Barrage of the Rand Water Board.

Fig. 27.—Vaal River Barrage of the Rand Water Board.

Vaal River barrage. The gates in all cases are thirty feet span and twenty-five feet high, which has been found to be the most convenient size. Gates of this type can be made of greater size, but in a country where all parts have to be transported on bad roads over long distances in wagons hauled by oxen or donkeys it is desirable to keep dimensions within moderate limits, so as to reduce the weight of individual parts. Where the cross sectional area of the waterway is limited it may be justifiable to adopt larger spans, but in most cases it is desirable to keep the units smaller and have more of them.

Stoney or Ashford balanced sluice gates with free antifriction rollers are highly efficient and very easily worked. Great care is, however, necessary in the erection of the gates, roller paths, grooves, and other items constituting the system. In most cases the superstructure carrying the lifting gear, main pulleys, etc., is made of light steel lattice work. During the Great War, owing to the high cost of material, this superstructure, and likewise the counterbalance boxes, were built in reinforced concrete, both in the case of the Rand Water Board barrage and at Lake Mentz. On the whole, steel work is, in the author's opinion, to be preferred, being generally cheaper and much quicker and simpler to erect.

Apart from their suitability for river trough reservoirs, sluices of this type are adapted to all cases where it is essential to avoid any considerable afflux in the case of a reservoir dam or diversion weir during periods of flood. In most cases it is possible by adopting this system in place of spillways to make the high flood level in a reservoir coincident with the full supply level.

OUTLET WORKS

A most important item in the design of all storage reservoirs is the device to be adopted for drawing the water from the reservoir and passing it through or round the dam.

In the case of earth dams, when it is necessary to take the outlet through the structure of the dam it is a particularly vulnerable feature, as unless great care is taken it may form a path for the percolation of water through the dam and cause its collapse.

This matter is so fully dealt with in most modern textbooks on the construction of storage reservoirs that it is unnecessary to deal with the subject further in this book. The construction

of high dams discharging very large volumes of water for irrigation purposes has introduced new complications into the design of outlet works, which were non-existent in former days when working heads were lower and the discharges withdrawn very much smaller.

For high heads large valves of the usual gate type are unsuitable. The great pressure on the gate makes them very difficult to open when they are entirely shut off. To meet this difficulty elaborate devices have been invented, in which the gate is first forced clear of its seating by hydraulic pressure before it is lifted by means of the head stock in the usual manner. The great drawback with valves of this type is that under high heads with a small lift the erosive action of silt-laden water discharging through the narrow opening at a very high velocity is very great and the deterioration of the valve may be very rapid.

With the older types of valves or gates serious troubles of two kinds were experienced: firstly, erosion of even the hardest and finest cut stone work and pitting and scoring of iron gate frames and tunnels below the gate due to vibration and "explosions." Troubles due to erosion were easily appreciated, and many improvements in design were effected, but the damage to the formation of a partial vacuum in the tunnel below the valve was not so quickly understood.

Much useful information regarding this subject is contained in an article in the "Engineering News-Record" of November 21, 1918, by James M. Gaylord, on "Some Experiences with Large Capacity Reservoir Outlets."

As the writer of the article rightly states, the difficulty is mainly the enormous amount of power involved in the discharge, amounting in some cases to hundreds of thousands of horsepower. If the discharge could be passed through water wheels a large part of the energy could be put to useful service, but the problem with a high storage dam is usually to discover a means of throwing this power away without damaging the structure.

One of the first high head gate problems was at the Rooseveld dam, where the maximum outlet discharge is 3,000 cusecs and the maximum working head is 226 feet.

The outlet installation consisted of two sets of three gates each—the downstream set for regulation and the upstream set for emergency closure to inspect or repair the regulating gates. Each gate closes an opening ten feet by five feet operated by a hydraulic device. The gate frame was provided with battered

bronze facing strips. The batter of the stationary face corresponds to that of the movable strip, and when the gate is closed these faces come into contact.

As soon as the gate is raised slightly the load is transferred to roller trains operating between the stationary and the movable track. Under maximum head (226 feet) the total water load on each gate leaf amounts to 800,000 lbs. and on each 4-inch roller the load is 13,000 lbs.

Water was turned through the gates for the first time in June, 1908. All the gates were opened wide and allowed to discharge freely for several months, during which time the head reached a maximum of ninety feet. In the following May an examination was made of the tunnel and gates, and damage to the outlet works was discovered. All the roller trains were broken or gone. Some of the deflection plates belonging to the walls above the gates had been carried away or loosened. The bronze gate seats were all damaged by blows from loose parts. Practically all of the nuts and bolts had been loosened, apparently by the vibration, and even the large nuts fastening the leaves to the piston rods were turned, one being four inches out of place. The concrete in the roof, floor, and piers was badly damaged; a piece about ten feet in diameter had fallen from the roof about twenty feet below the gates. The concrete floor, extending twenty feet below, had been undercut several feet back towards the piers.

Temporary repairs were made, and in an attempt to supply an urgent demand for water the gates were operated for two days. At the end of this time it was found that the entire concrete floor and much of the concrete in the piers had been washed out, and in one case a passage made under the sill of a gate.

The above quotation is given to show the unsatisfactory nature of the sliding gate design, where enormous volumes have to be dealt with even under the comparatively low head of ninety feet, and similar experiences were met with in the case of the sliding gates at the Pathfinder dam. It was concluded that the damage had resulted not only from the scouring action of the high velocity discharge, but likewise from the violent disturbances caused by the obstruction which the gate leaf in a partially open position offers to the flow of water. These experiences led to the evolution by the United States Reclamation Service of a highly efficient form of balanced valve, known as the "Ensign Valve."

These devices consisted essentially of a cylindrical piston with one end pointed to guide and regulate the discharge, the other end being provided with a bull ring of larger diameter fitting a stationary cylinder, in which it moves. The cylinder is attached to the circular base casting by a supporting grillage, through which the water flows in a radical direction. Striking the pointed end of the plug, the flow is turned through an angle of 90 degrees and discharged axially through the base casting into the circular outlet pipe.

There are various forms of efficient balanced valves of this type which are suitable for irrigation reservoirs discharging large volumes of water under moderate to high heads. Whilst the substitution of these balanced needle valves largely overcame all the difficulties in connexion with damage to the valve itself, new troubles were experienced to points some distance below the valve. It was found that immediately below the valves the cast-iron pipes began to show serious wear and cavitation after two or three seasons of operation. This action, which is attributed to the formation of a vacuum at this point, continued until the pitting extended entirely through the metal and for some distance into the surrounding concrete ; and this trouble has been overcome by the admission of air to the critical point in the jet and by installing throat pieces to approximate the shape of the contracted jet. Whilst it was not possible in the case of the earlier structures, such as Rooseveld and Pathfinder dams, to make the design entirely satisfactory, this has been achieved in the case of the Arrowrock dam, where, after operation of the valves for three seasons under heads up to 130 feet, it was found that the control of the discharge was perfect and no damage had occurred either to valves or outlets.

The experience thus gained with these large American works is of the utmost importance to irrigation engineers, and this difficult problem may now be considered to have been satisfactorily solved.

CHAPTER VII

SILT

SILT carried in the waters of rivers either in suspension or rolled along the bed is one of the most fertile sources of trouble to the irrigation engineer. Erosion takes place in every river catchment, though it varies greatly in degree. Where the country is arid and bare of vegetation and the rain falls chiefly in sharp thunder-showers erosion is heavy. In areas where the rainfall is greater and more regular the surface is clothed with more dense vegetation, the contour of the country is less harsh, the surface run-off is slower, and the erosion very much less, sometimes almost negligible. Generally speaking, the lower and more erratic the rainfall, the greater the erosion, and hence the dirtier the water in the stream.

In South Africa this rule applies in a marked degree. Furthermore, over a large part of the drier regions, such as the Karroo, the exposed rocks are frequently very soft and weather readily. Rivers rising in or flowing through Karroo strata, especially the Beaufort beds of that formation, are invariably very muddy, the rocks eroded being shales, sandstones, and decomposed dolerite.

As has been previously stated, most Karroo rivers are at present cutting fresh or enlarged channels for themselves in the deep alluvial deposits of former times. This process, combined with the general desiccation which is now in progress throughout the drier parts of the Union, causes all these rivers to carry heavy burdens of silt.

In the northern portions of the Union the country is of more gentle contour and is generally well covered with grass or bush, and the rocks are very much harder and more resistant to weathering. Transvaal and northern Free State rivers are therefore much cleaner, and most of the material carried is heavy and rolled along the beds.

The coastal regions likewise produce comparatively little silt, though in many cases they transport a heavy burden of sand and gravel.

So far as irrigation works are concerned the silt problem is confined to the Karroo or plateau area of the Cape and Free State provinces. Whilst silt is an unmitigated nuisance to the engineer, it is, generally speaking, of great value to land on which it is deposited. Many of the richest agricultural regions in the world owe their fertility to annual silt deposits which the land receives by means of irrigation furrows or inundation. Land watered by an irrigation system carrying rich silt in its waters will continue to yield heavy crops indefinitely without artificial fertilizers. In many cases it is possible to convert poor, barren soils into very rich ones by systematic irrigation with water containing rich silt.

Clear water is never so good for irrigation as turbid water, provided the silt is not too fine. In South Africa, for example, the richest agricultural lands are located on the deep alluvium of the Karroo rivers, and as these are generally irrigated with silt-laden water the land does not become impoverished. In the Transvaal, on the other hand, the natural alluvial soil is less fertile and receives no beneficial silt when irrigated.

It is obvious, therefore, that to the fullest extent possible an irrigation system should carry water with its full burden of silt on the land without allowing it to deposit the greater proportion of its load in the channels. All silt deposited in canals is merely an obstruction which reduces their carrying capacity, and has to be removed from time to time at great cost. For years past, therefore, irrigation engineers have been endeavouring to design canals in such a manner that the velocity of the water flowing in them shall be such as to maintain as much silt in suspension as is the case in the parent stream from which the water is taken into the canal, and to deliver the turbid water on to the land to be irrigated without allowing any silt to be deposited on the bed of the canals. In the case of the enormous canal system of Upper India very considerable success has been achieved in this direction. In the case of most South African irrigation canals, which take water from intermittent rivers during flood time only, the problem is not capable of solution in this simple manner.

The silt problem is an entirely different one when we are dealing with storage reservoirs. A reservoir fed by the muddy

waters of a river must of necessity silt up and in course of time become useless, unless the silt can be trapped before the water reaches the reservoir, or unless some provision is made to periodically scour the silt out of the reservoir.

The two cases must therefore be dealt with separately. Before dealing with the practical side of the question it is necessary to understand clearly the nature of silt and its behaviour in running streams or canals, and in lakes or reservoirs into which muddy water is discharged. The subject has received considerable attention, more especially in northern India, Switzerland, and in the United States of America. In the two former countries the silt problem has received more scientific study than elsewhere.

One of the earliest and most important discoveries was made by R. G. Kennedy, of the Punjab Irrigation Service, who proved that the power of water in an open channel to carry silt in suspension decreases as the depth of water increases. Every particle in suspension whose specific gravity is greater than unity tends to sink. It is kept in suspension by eddies which are produced on the bed. The existence of these eddies can be noted in any stream of turbid water. As these eddies reach the surface they produce the familiar boiling effect.

The silt particles are prevented from sinking by the upward components of velocity in these eddies.

If V is the velocity of the water and D its depth in a stream, the force exerted by the eddies generated on one square foot of the bed varies directly as the velocity.

With an average charge of silt its weight in a vertical column of water with a base of one square foot varies as D. Hence the power of a stream to support silt in suspension varies directly as some power of V and inversely as D. Thus the velocity which will maintain the silt in suspension may be expressed by the equation

$$V_o = cd^m,$$

the power m being less than unity.

On the basis of this formula it has been possible to construct hundreds of canals and distributing channels in northern India which maintain themselves more or less free from silt. This aspect of the subject will be referred to later on.

A close study of the problem by French, Swiss, and German engineers has established, *inter alia*, the following facts:

A particular velocity of water in a stream will only transport

a certain definite quantity of silt; once this point of saturation is exceeded silt deposit will take place. This is accounted for by the Kennedy theory referred to above.

The presence of silt in suspension has the effect of reducing the velocity of flow, and the greater the load of silt the lower will be the velocity. Silt-laden water becomes in fact viscous to some degree, and beyond a certain limit of silt charge the water would cease to flow unless the velocity were increased. This interesting fact has an important bearing on the gauging of a stream or channel, as, with the same cross section and depth of flow, the velocity, and consequently the discharge, will be greater when the water is clear than in the case where the water is charged with silt.

These facts have been demonstrated by actual gauging of Swiss streams, and from these it is apparent that the effect of silt on the velocity is very considerable. A discharge curve will thus not be equally correct for streams carrying clear and muddy water.

When the velocity of water in a channel is not very great the temperature of the water has a considerable effect upon the capacity of silt to remain in suspension. Experiments by Sudry, in France, have shown that with particles 0·025 cm. in diameter the velocity of fall in water increases 100 per cent if the temperature of the water increases from 10° to 35° Centigrade. With particles 0·043 cm. the velocity of descent increases 50 per cent for the same temperature rise. The velocity with which particles of silt will fall is greater in clear water than in water carrying a considerable quantity of colloidal mud in suspension.

It is obvious that silt-transporting capacity of rivers and streams will vary enormously according to the gradient of the channel and other conditions. Torrents in mountainous country with very steep grades are capable of moving enormous masses of material, including large boulders. As rivers continue in their course their grades become flatter, their velocities smaller, and their powers of transportation less. This is shown clearly by the character of the alluvial deposits as one proceeds down the course of many large rivers. In the upper reaches deposits of boulders, gravel, and coarse sand predominate. Proceeding downstream the material becomes less coarse, and in the lower reaches and deltaic zones the silt is generally exceedingly fine.

Where irrigation or power works are located in the torrential zone the problem consists mainly in keeping the heavy rolling

silt and gravel out of canals taking off from such streams. This can generally be effected by various structural devices. Reservoirs are not generally located at such sites owing to the small capacity obtainable.

Cases sometimes occur where a reservoir is fed by a canal taking out of a dirty river. For such cases it is possible to trap nearly all the harmful solids out of the water near the off-take of the canal and deliver only comparatively clear water into the reservoir. Such instances are, however, uncommon, and in South Africa they are unfortunately very rare.

In South Africa the usual case is for irrigation works, whether reservoirs or diversion weirs and canals, to be located along portions of river systems where gradients range from ten to thirty feet per mile, and velocities, especially during flood time, are high enough to transport large percentages of sand and silt.

It is therefore desirable to know what amount of solid matter is actually transported by such rivers and streams.

Whilst the power to carry solids is a function of the velocity and shape of the channel, and is the same everywhere, the actual volume of the solids carried varies with each stream, according to the character of the country which is being eroded by the river. Hence it is necessary to determine this volume by experiment in each individual case.

We are here faced with serious practical difficulties. It is only possible to determine the total amount of solid matter transported by a river by analysing samples of the water. Such samples under the best of conditions only show what amount of silt is carried in suspension. No satisfactory method has ever been devised for determining the actual amount of heavy material rolled along the bed. This heavy material is, however, small in comparison with the volume of solids carried in suspension. Nothing is more difficult than to abstract samples which may be taken as representative and used for the purpose of calculating the total volume transported during a given time, per diem, per annum, or for the duration of a flood.

Single samples of water taken from a river in bottles or boxes are quite useless. Such samples are not even approximately representative of the water at the particular spot and time at which the sample is taken, as the insertion of the bottle or box into the running stream entirely upsets the normal condition of flow. It may create eddies and cause a concentration of silt at the particular spot, or it may have the contrary effect.

The concentration of silt in suspension varies throughout the cross section of the river and also longitudinally. Innumerable experiments have been made in India, Switzerland, and elsewhere to endeavour to establish the relationship between silt concentration and depth of water, but it is evident that no rule or law can be established. In some cases the greatest concentration is found at or close to the surface ; in others at half depth or near the bottom. On the same river at the same site the region of concentration in relation to depth will vary considerably from day to day, possibly according to the nature of the silt carried at the time, and to changes in the irregularities of the bed.

The concentration varies to some extent laterally, according to the irregularities of the bed and the changes in mean velocity at different points of the cross section. It is clear, therefore, that at any given time a considerable number of samples must be taken at different points of the cross section, both horizontally and vertically, in order to obtain a mean value of the quantity of silt carried in suspension.

In a perennial river the water is normally clear, or contains but little solid matter. When in spate the water becomes muddy and carries its maximum load of silt in the early stages, and the greatest concentration of solids may or may not coincide with the peak of the flood. During the latter stages of the flood the amount of silt carried rapidly decreases, and after a time the water becomes clear again. In the case of the South African rivers in the Karroo, which are practically all intermittent, the clear flow is negligible. The discharge consists of a series of freshets or floods with long dry periods intervening. Under such conditions the amount of silt carried in suspension is a much greater percentage of the total annual flow than is the case with streams having a considerable perennial flow. The total volume of silt carried by these rivers is likewise most difficult to estimate, and would necessitate experiments on a very large scale continued over a lengthy period of time. Unfortunately, no such experiments have as yet been carried out in South Africa.

When dealing with this question experimentally the immediate result obtained is the percentage of dry weight of silt got from a cubic foot or other capacity measurement of the water sampled. The practical problem, however, is to estimate the percentage of space in the reservoir which would be occupied by the silt contained in, say, a cubic foot of water, if it were all

deposited, due regard being had to the mean consolidation of the deposited silt when the depth thereof is considerable.

Davis and Wilson, in their book on "Irrigation Engineering," quote four of what they consider the most reliable results, which give seventy-seven pounds as the average dry weight of a cubic foot of silt deposited in a reservoir. C. H. Warren, Irrigation "Circle Engineer," in an article on "Silt in the Flood Waters of the Great Fish River," contributed to the "South African Irrigation Magazine" (Vol. I, No. 2, Jan., 1922), gives the weight of one cubic foot of artificially dried Fish River silt as 52·8 lbs., the sample being taken from a fresh and deep deposit of silt. The determination of this weight is largely a matter of judgment, which accounts for the great variation in the results of different observers, quite apart from the fact that there are considerable differences in the character of silts of individual rivers, and at different times in the same river.

Silt when first precipitated from the water in which it has been in suspension forms a light and bulky ooze. This is specially the case where, as in Karroo rivers in South Africa, a large percentage of the silt is of a very fine and even colloidal character. Such silt, when freshly deposited on a reservoir bed, forms a jelly-like substance. In course of time the superincumbent weight of succeeding deposits gradually compacts the lower layers of silt, which eventually will attain the density of a medium loam or a heavy clay. In this condition its specific gravity may be as high as 2·0, but may certainly be assumed to be 1·60, or 100 lbs. per cubic foot. The final bulk of the silt will therefore be about two-thirds of the volume of water to which it was originally equal, and the volume of the deposit compared with that of the water in which it was suspended will be, approximately, two-thirds of the above proportions by weight.

When making experiments to ascertain the amount of suspended silt in a sample of water care must be taken to place clearly on record on what basis the silt content is calculated, i.e. whether on the basis of dry matter obtained by completely evaporating all the moisture, or on the basis of displacement after the silt has been allowed to settle. Much confusion has arisen owing to the fact that different observers give results according to different methods of reckoning, and we have results recorded as percentages of the volume of water flowing in a certain time, as proportion of dry silt to water by weight, in parts per million, etc.

Various authorities quote figures for many rivers in different parts of the world. The fullest information is available in respect of European rivers, and in some cases (for example the Drance, in Switzerland) continuous diurnal records are given for every day for several years. Interesting as these data are they have no application to conditions such as exist in South Africa, more especially to intermittent rivers.

In the article by Warren quoted above, and in other articles in the " South African Irrigation Magazine," the following data relating to silt in South African rivers are given, but as these are based upon single examples obtained by crude methods they can only be accepted as a very rough approximation :

TABLE XXVI

Experimental Silt Content of certain South African Rivers

River	Period	Percentage by Weight	Percentage by Volume
Great Fish River (Cradock)	1919–20	2·11	2·48
Do.	1920–21	3·98	4·7
Do.	1921–22	1·36	1·6
Do.	Maximum observed percentage, Feb., 1921	9·87	—
Great Fish River (Middleton)	1920–21	4·32	5·11
Do.	1920–21 (Maximum)	7·98	—
Vlekport River	1920–21	5·06	6·00
Do.	1920–21 (Maximum)	11·24	—
Brak River (Grass Ridge dam site)	Sept., 1921	0·76	0·90
Tarka River (Tarka dam site)	Sept., 1921	1·93	2·30

In Vol. I, No. 3, of the " South African Irrigation Magazine " the following information is given regarding silt in certain South African rivers. Referring to the Orange River at Orange River Station no silt measurements were made before the middle of March, 1921, prior to which the following maximum flood

discharges had been recorded: 15,000 cusecs in October, 1920; 17,000 cusecs in January, 1921; 31,000 cusecs in February; and 20,000 cusecs on the 12th March. On March 18th, with a discharge of 16,500 cusecs, the silt content was 0·44 per cent; 0·63 per cent on the 23rd with a discharge of 26,000 cusecs; and a maximum of 0·74 per cent was obtained on the 30th March with a discharge of 31,000 cusecs. Observations made in 1920 gave the following results after a period of about two months without floods:

Date	Discharge in Cusecs	Silt Content
January 23 ..	4,000	0·68 per cent
January 30 ..	7,600	0·88 ,, ,,
February 3 ..	67,000	0·92 ,, ,,
February ..	235,000	1·04 ,, ,,

Observations on the Vaal River, near Kimberley, show that the maximum silt content during the years 1918 and 1919 occurred on January 9, 1918, when it amounted to 0·37 per cent, the discharge at the time being 78,000 cusecs.

The Vaal River is not a Karroo river, and whilst always slightly turbid is essentially a clean river.

One of the dirtiest rivers so far systematically investigated is the Rio Grande, in New Mexico (U.S.A.), and the mean value obtained over a period of 16 years from 1897 to 1912 was 1·66 per cent of volume. These percentages are based upon a dry weight of 53 lbs. for a cubic foot of wet sediment supposed to have been deposited in a reservoir.

While quantities of silt ranging up to as much as 4 per cent per annum are recorded, the maximum concentration of silt for short periods of time during floods may be much greater and may reach as high as 10 per cent. It is very doubtful whether in most rivers such concentration could be much exceeded even when in high flood, as the liquid would tend to become viscous and cease to flow freely. Of 19 cases quoted by Buckley in the "Irrigation Pocket Book" giving maximum amounts of silt gauged in various rivers in India, in Europe and in the Nile, the highest concentration measured was 4 per cent by weight in a river in Bengal exposed to strong tidal action. The next highest is the Sutlej in the Punjab, with 3 per cent. The silt content is given in grammes of silt per cubic metre of water.

The specific problems relating to silt will now be dealt with.

THE SEDIMENTATION OF RESERVOIRS

When dealing with reservoirs, whichever way we may express the relationship of silt to water, the conclusion arrived at is that if *all* the silt in suspension or rolled along the bed by the flow were deposited in a reservoir formed by damming the river the reservoir would completely silt up in a certain number of years.

In some cases the period required for complete silting up is alarmingly short. In the case of small farm reservoirs or weirs across rivers it is common knowledge that complete siltage does actually occur in a very short time.

In the case of the Great Elephant Butte reservoir on the Rio Grande it was demonstrated that if a reservoir of 300,000 acre feet capacity had been constructed in 1897 it would have been filled with sediment in about fifteen years.

In that particular case the reservoir has been given such an enormous capacity that the required storage will not be encroached upon for about forty or fifty years, and thereafter reliance will be placed upon the enlargement of the dam and on the construction of subsidiary reservoirs higher up the catchment. This appears rather a hopeless proposition and one which could be followed with difficulty in many other cases.

There is a great tendency to be unnecessarily pessimistic about the silting up of large reservoirs. It is wrong to assume that all the silt transported by a river into a reservoir will remain there permanently in the form of deposit. Leaving out of account for the present all special devices for scouring silt out of a reservoir, there is no question but that in the case of all reservoirs from which there are large withdrawals of water, as is the case with irrigation, flood regulation and some large water power undertakings, a very large percentage of the silt which is transported into the reservoir is naturally taken out of it again through the outlet sluices, spillways, or surplus water sluices.

When silt-laden water in a river discharges into a lake or into the sea the velocity of flow is suddenly checked, and the bulk of the rolling sand and all the heavier particles of suspended silt are deposited. Deltaic conditions are established. In the case of natural lakes these delta formations are seen to great advantage at the head of all the large Swiss lakes. Many of these deltas have been very closely studied for many years and

their regular extension into the lakes has been measured. In the case of a storage reservoir which was maintained more or less full identical conditions would be established. As, however, the level of the water in most artificial reservoirs is constantly changing, the deltaic conditions become modified and the accumulation of gravel, sand, and heavy silt is spread over a greater length of the original river channel than would be the case if the water level in the reservoir were constant. After a time, however, a broad bar of sediment is formed at the head of the reservoir which effectively checks the velocity of the water coming in, and deposition of silt, etc., is thereby encouraged at the upper end irrespective of what the water level in the reservoir may be. Under such conditions the tendency will be for this bar or delta to move slowly forward towards the dam.

The heavy and most undesirable material will, for a long period of time, be concentrated at the extreme upper end of the reservoir in comparatively shallow water. Whilst sand and heavy silt is thus accumulated far from the dam, the lighter particles of silt are carried into the reservoir, and if the water remained still for a lengthy period the whole mass of this fine silt would be deposited on the floor of the reservoir.

In the case of Karroo rivers the fine and medium silt particles, which are carried into the reservoir and reach the dam, form a very large percentage of the total volume of solids transported by the rivers feeding the reservoir. Dr. Stead, in a paper on "Agricultural Aspects of Irrigation," read before the South African Society of Civil Engineers (Vol. XVIII, 1920, of the "Proceedings"), gives the following mechanical analysis of silt carried by one of the large Karroo rivers:

Nature of Material	River in Full Flood	In a Canal taking off from the same River and at the same time
Coarse sand	*Nil*	*Nil*
Fine sand	16·46	6·00
Silt	19·90	4·00
Fine Silt	24·13	34·30
Clay	29·55	46·10
Moisture	0·98	1·25
Material soluble in 2% HCl	3·10	2·40
Loss on ignition	5·20	6·30

The total suspended solids carried by the river at the time was 5·04 per cent and 1·46 per cent in the canal.

Warren, in his article on silt quoted above, states that in the case of the Great Fish River the samples taken showed that on an average fifty per cent of the silt was "solid," i.e. settled in still water in a comparatively short time, the remainder was flocculent. Settlement by gravity ceased altogether after three months' standing, the water still remaining turbid.

The figures quoted above are typical of most Karroo rivers, and show that a very large percentage of the silt is of a very light character. With a very little agitation a great deal of this silt will be kept in suspension, and the author has been impressed by the high silt content of water in a reservoir with a large surface area which had been drawn fairly low during an even moderate wind, even though the water had been standing in the reservoir without any replenishment whatever for many months. During August, September, and October throughout the Karroo winds are high, and as this is the period of most intense demand for water for irrigation withdrawals are on a big scale. It follows, therefore, that a large percentage of the silt in the reservoir is drawn off through the irrigation outlets. In the case of several large reservoirs in South Africa constructed within the past ten years there is no appreciable accumulation of silt near the dam or outside the original river channels.

One danger must, however, be guarded against. Karroo rivers carry a certain amount of salts in solution, some of which are strong coagulents, and if water is impounded for too long a period evaporation may cause a sufficient concentration of these salts to bring about the coagulation of the fine silt and its deposition on the floor of the reservoir in a more solid and resistant form.

The self-cleansing of reservoirs, as above described, is not confined to South Africa. In the case of the Burrin Juck reservoir in New South Wales, the silting problem has been under investigation for many years. Some interesting facts have been brought to light in the official reports of the New South Wales Government (P.W.D.). In Table XXVII are shown the estimated solids in cubic feet for each of seven years.

Table XXVIII shows for the year 1916–1917 for each month the solids in the inflow and those in the outflow. During some months the concentration of solids in the outflow is greater than that in the inflow, or, in other words, scour took place.

TABLE XXVII

Burrin Juck Reservoir, New South Wales.—Estimated Silt Content of Water Entering Reservoir.

Year	Discharge in Million Cubic Feet	Estimated Total Solids in Cubic Feet
1910–11 ..	35,193·300	5,431,229
1911–12 ..	19,247·800	4,427,613
1912–13 ..	36,598·100	14,133,754
1913–14 ..	31,609·970	29,085,227
1914–15 ..	16,800·016	6,093,835
1915–16 ..	44,511·029	17,434,402
1916–17 ..	95,363·505	35,372,030

In this year it was computed that out of a total volume of solids in the inflow amounting to 35,372,000 cubic feet, no less than 23,522,000 cubic feet of solids were contained in the outflow; thus only 11,580,000 cubic feet would have been actually deposited in the reservoir, which has a total storage capacity of about 33,000,000,000 cubic feet.

To deal with surplus water a reservoir dam may be provided with an ordinary byewash or waste weir at full supply level, with a depressed spillway controlled by shutters or large sluice gates, the tops of these being at full supply level, or with sluices through the body of the dam working under pressure. The last named are essentially scouring sluices and will be dealt with later. The first two types, being primarily devices for disposing of surplus water, will first be dealt with.

When reservoirs are large in comparison with the run-off available, and surplus water will only rarely be discharged over the spillway, the tendency to silt up will be greater than in cases where the spillways are frequently in action and are working under considerable heads.

For the usual conditions existing in this country the author is strongly in favour of substituting large sluice gates, say twenty to twenty-five feet deep for the usual spillway at full supply level. In most reservoirs of moderate size the bulk of the storage capacity is in the top twenty feet of the reservoir, and if eventually the basins silt up to the sill level of the gates, no material loss of storage is caused thereby. If surplus water has to be discharged from the reservoir the opening of a sluice

TABLE XXVIII

Burrin Juck Dam.—Analysis of Silt Record for 1916–1917.

Month	Inflow				Outflow			
	Grains of Solids per 600 cc.	Parts per Million	Inflow in Millions Cubic Feet	Estimated Solids in Cubic Feet	Grains of Solids per 600 cc.	Parts per Million	Outflow in Millions Cubic Feet	Estimated Solids in Cubic Feet
July	4·277	449·085	9,741·888	4,374,158	1·696	178·080	8,380·638	1,492,818
August	1·684	176·820	12,567·911	2,224,536	2·144	225·120	9,838·161	2,213,550
September	2·892	303·660	6,526·388	1,983,904	2·949	309·645	8,108·388	2,513,480
October	5·047	529·935	30,397·678	16,110,940	3·633	381·465	27,760·428	10,576,560
November	1·447	151·935	10,752·626	1,634,456	2·033	213·465	10,694·375	2,277,822
December	3·442	361·410	8,534·097	3,080,774	1·874	196·770	8,494·999	1,671 155
January	5·328	559·440	2,456·756	1,373,463	1·385	145·425	2,612·340	378,740
February	5·132	538·860	1,635·099	879,630	1·242	130·410	1,557·432	202,410
March	3·220	338·100	1,225·386	414,050	1·508	158·340	1,186·553	187,546
April	1·353	142·065	909·823	129,220	1·934	203·070	890·406	180,670
May	2·297	241·185	3,049·719	735,050	1·529	160·545	1,916·759	308,637
June	3·194	335·370	7,566·094	2,534,610	1·872	196·560	7,629·094	1,502,913
TOTALS	3·5325	370·918	95,363·505	35,372,030	2·515	264·086	89,069·574	23,522,016

gate having a sectional area of thirty feet by twenty-five feet for even a short period of time has a very great scouring effect and will induce a strong current right through the reservoir, whereas allowing the same surplus volume of water to spill over a waste weir in a thin film for several days would have no scouring effect whatever.

A really large flood discharge through such sluices will move enormous volumes of silt. The ancient reservoirs of Spain, constructed by the Moors centuries ago, which would silt nearly to full supply level within a short period of time, have been kept clear of silt by much more primitive means up to the present time.

With the fine silt contained in the Karroo rivers it is of great importance to avoid its settling as much as possible, and this provides yet another reason in favour of providing sluice gates in place of spillways. The action of such sluice gates, apart from agitating the loose sediment in the floor of the reservoir, has a general freshening effect on the mass of water stored. All water derived from Karroo rivers contains certain brak salts; and if water remains quiescent in a reservoir for a considerable period whilst heavy evaporation is going on these salts tend to assume a harmful degree of concentration. The worst offenders in this respect are sodium carbonate and calcium and magnesium bicarbonates, which, if present in appreciable quantity, act as coagulents on the fine argillaceous material in suspension, causing its precipitation, which is just the thing we desire to avoid. The opening of a single sluice gate during a small freshet will effect a more rapid replacement of brak water by fresh water than would be the case with a spillway skimming a thin film of water from the surface.

Sedimentation of reservoirs cannot, under conditions generally prevailing, be wholly prevented.

Where very large volumes of surplus water are annually available complete cleansing by means of a large battery of sluices placed at the bottom of a dam is possible.

A successful modern example of this method of silt scouring is to be found in the Bhatgarh dam (Lake Whiting) in the Bombay Presidency in India. This dam is 103 feet above the river bed and impounds 5,313,000,000 cubic feet. The dam is pierced at a level of 80 feet below the full supply level by 15 sluices, each 8 feet by 4 feet, through which all the early floods, which carry a very heavy load of silt, are discharged. The

gates are then closed and the reservoir is allowed to fill. In this particular case it is possible to pass 30,000,000,000 cubic feet through the sluices before it becomes necessary to close the gates in order to impound 5,313,000,000 cubic feet. The Assuan dam on the Nile is likewise an example of this type.

In the case of reservoirs created in the troughs of rivers that problem is satisfactorily solved by the construction of open dams or barrages, of which the Rand Water Board's barrage on the Vaal River near Vereeniging and the Olifants River barrage, which were described in the chapter on conservation, are typical examples.

Scouring by means of a limited number of comparatively small pressure sluices are of doubtful utility in the case of large reservoirs, though, under special conditions, they have been effective in maintaining reservoirs comparatively clear of silt for a very long period. As there are several classic examples of this type about which little is known, this particular system, known as the Spanish method of scouring silt out of reservoirs, will be treated in some detail.

One of the earliest methods adopted for scouring silt deposit out of reservoirs was devised by the Moors in south-eastern Spain, and the devices designed by these enterprising irrigation engineers are still in operation to-day; and though crude in many respects, they have been effective in maintaining the efficiency of several reservoirs fed by heavily silt-laden waters for some centuries.

The process is carried out by means of scouring galleries or sluices. It is carried out more or less regularly at the dams of Tibi (in Alicante) and at Elche. In the case of the Tibi dam it is now considered necessary for the silt to accumulate for a period of ten years, during which time the depth of silt at the dam varies from sixty to sixty-six feet. To clear the reservoir it is necessary to have a depth of from ten to thirteen feet of water over the silt. The bottom scour tunnel is then opened and the water bursts through the silt, a large portion of which is then carrried away at once. The balance of the silt is to a considerable extent washed out by the small permanent supply of water passing through the reservoir when the dam is empty. During the first stage the mass of silt near the dam is all washed out, leaving the remainder standing with high, steep faces, with gullies or ravines running into it. This remaining mass of now waterlogged silt sloughs slowly towards the centre of the basin, and is then

FIG. 28.—View of reservoir basin of Tibi Dam, Alicante, Spain, taken a few days after silt-scouring gallery had been opened.

carried away by the small stream of water flowing through the reservoir. The author visited the Tibi dam in December, 1908, a week after the scouring sluice had been opened after an interval of sixteen years. At the moment the sluice was opened the depth of silt at the dam was 79·5 feet and the depth of water above it was 8·6 feet.

The total height of the dam is 146·6 feet. The reservoir was empty, with a small quantity of water pouring through the scour sluice fully charged with silt. The reservoir was entirely clear of silt for a distance of about 300 feet above the dam. Upstream of this point the clearance was only partial, but was proceeding by sloughing as described above. Fig. 28 gives a view of the reservoir basin towards the end of the scouring operations.

The river below the dam flows in a narrow gorge, and the evidences which the initial scouring process had left behind were somewhat alarming. The gorge was full of mud, and very considerable damage had been caused by the torrent. The Spanish reservoirs are located on streams with steep gradients, and the capacities are small in proportion to the heights of the dams. These conditions make the scouring operations possible with the comparatively small galleries or sluices provided. With reservoirs extending back a much greater distance from the dam the cleansing would be of a very partial nature only.

According to ancient regulations the scouring sluice should be opened every four years, and if this rule were adhered to the cleansing would, no doubt, be more effective than after a period of sixteen years. The scouring gallery of the Tibi dam consists of a culvert through the centre of the dam at the bottom. It is 5·9 feet wide and 8·9 feet high at the upstream portal, but is larger lower down. The gallery or culvert is blocked near the upstream face by a bulkhead composed of vertical wooden beams about twelve inches square, which are fitted into grooves in the roof and floor. The central beam, which is the key, is shorter and is not held by the grooves. It is held in position by a second series of wooden horizontal beams, which are placed immediately below the vertical series and are supported in grooves on each side. This horizontal series does not extend to the top of the gallery, but only high enough to hold the vertical key beam securely in position. Finally, two vertical beams supported by struts complete the block in the gallery.

The silt in the gallery above the block sets very hard. When the time has arrived for opening the sluice a man enters the gallery and removes the timber balks carefully, or loosens them with an axe. If any movement in the silt block is noticed he must escape as fast as he can. If the silt block continues to hold, provision must be made for breaking it down by means of a rough device resembling a jumper drill, which is worked through a circular hole, penetrating the structure of the dam from the crest to the roof of the gallery. The opening of the sluice is of a distinctly exciting character. A graphic description thereof is contained in Aymard's classic book, "Irrigations du Midi de l'Espagne."

The method above described has been elaborated by artificially agitating or stirring up the silt by means of compressed air and by means of mechanical devices, but it is reported that no practical results have been obtained.

The Spanish engineers evidently have faith in this method, as it has been adopted, with improvements involving less risk to the operator, in the case of dams constructed in recent years.

The famous old Puentes dam, which failed in 1802, was rebuilt in the early 'eighties of the last century, close to the site of the original structure. The new dam embodied the typical scouring devices of the earlier Moorish dams, but with numerous improvements. There are three scouring galleries at the bottom of the dam. Two of these near the centre are approximately 6 feet 6 inches high and 4 feet wide, whilst the sluice nearer the right abutment is 4 feet 10½ inches high and 3 feet 3 inches wide. All galleries were controlled by gates worked by hydraulic power. About 1900 the working of this dam was carefully investigated by a commission of engineers. A portion of their report is taken up with a discussion of the troubles experienced with the gates when discharging under high heads. In this respect experiences were identical to those experienced at a much later date at the Rooseveld dam, which have been described at the end of the chapter on conservation. The commissioners refer frequently to the destructive effects of "cannon shots," but go astray when trying to explain these phenomena.

The most interesting portion of the report deals with the efficiency of the sluices as silt-scouring devices. The report deals in considerable detail with the design of the galleries, gates, and the manipulation thereof, which, though of general interest, will not be further commented upon here. The details of the

silt scouring effected from time to time are, however, of great interest.

The dam is located in a gorge of the River Guadalentin, about one kilometre below the confluence of the Rivers Velez and Luchena. The maximum height of the dam above ground level is 156 feet. The original capacity of the Puentes reservoir was approximately 953,000,000 cubic feet. The catchment area is approximately 800 square miles, composed of rough, mountainous country, largely bare. The river is subject to very violent floods, and enormous quantities of silt are carried in suspension.

Three specially large floods are referred to in the report.

TABLE XXIX

Date	Total Volume of Flood in Million Cubic Feet	Maximum Intensity of Discharge in Cusecs
Sept. 11, 1891 ..	423·6	66,700
Sept. 12, 1897 ..	529·5	17,756
June 26, 1900 ..	1,588·5	45,714

Whilst it will be seen that the maximum discharges are very high—the greatest recorded intensity of run-off reaching the high figure of 83·5 cusecs per square mile from an 800 square mile catchment—the normal dry weather discharge is only about 15 cusecs.

The silt content of the water during high floods is stated to reach the extraordinary high figure of twenty-five per cent. The accuracy of this appears very doubtful.

The periodical cleansings have to be fitted in so far as is possible with irrigation requirements, and twenty-four days was found to be the maximum period which it was possible to devote to the process. Experience showed that for efficient cleansing a permanent flow of about a hundred cusecs is necessary during the second stage of the process after the reservoir has been emptied. For this reason the time for scouring must be very carefully chosen, i.e. during a period of the year when the irrigation demand is not very great, and directly after a thunderstorm, or, better still, after snow has fallen over the catchment. The actual sequence of events is described by the commissioners as follows: When the galleries are opened, in order to start the cleansing, the initial discharge is heavily charged with silt, which

is deposited near the mouth of the galleries. This first stage lasts from ten to fifteen minutes. The discharge then becomes clear again, and remains so until a large amount of water has been drawn off. The third stage now commences, during which the wall of silt commences to slough, forming deep ravines, extending upstream with the reservoir. The silt becomes detached in great blocks, which are transformed into big balls, which are rolled along with the current towards the gallery openings. This stage requires careful handling, as if the lumps and balls of silt are large they tend to block up the galleries. To avoid this the discharge is checked somewhat when necessary, so as to allow the lumps to dissolve or disintegrate. It was found that if a gallery became choked with pure silt a head of from sixty-five to eighty feet of water was generally necessary to clear it. It is clear from the report that the blocking of tunnels with silt and even rocks is a constant source of trouble. The greater the interval between successive cleansings the greater the danger of choking. Thus in 1893 great trouble was experienced, there being a nine years' accumulation of silt in the reservoir at the time. Three other cleansings, the interval between which did not exceed one year, took place without any incident worth mentioning.

In view of the troubles experienced in opening the gallery gates under high heads the commissioners recommend that the head of water when operations start shall not exceed from sixty-five to eighty feet.

The following are records of some complete emptyings of the reservoir:

TABLE XXX

Date	Duration of Operation	Discharge Flowing into Reservoir. Cusecs	Silt Scoured Out. Cubic Feet	Remarks
January, 1893 ..	23 hours	106	28,805,500	Work stopped due to choking of galleries
December, 1894	23 days	282	158,750,000	
February, 1898 ..	15 hours	247	—	Galleries blocked by two large rocks
November, 1898	10 days	141	36,052,250	
November, 1900	24 do.	141	158,000,000	

The conclusion arrived at as a result of the inquiry was that the problem of scouring silt by these methods had not yet been solved. The authorities in charge are said to have done their utmost to keep the reservoir clean, and during the period 1893 to 1920 actually scoured out 371,500,000 cubic feet of silt; yet at the close of these operations there still remained 247,000,000 cubic feet of silt in the reservoir, or about a quarter of its original capacity.

It is considered that with certain improvements in design and method of working the silt problem can be, at any rate, partially dealt with for a very long period of time in the case of irrigation reservoirs, and for an indefinite period in the case of reservoirs constructed for flood regulation purposes only.

In the design of the Buseo dam, on which construction was commenced in 1903, provision for similar silt-scouring galleries was made. This dam has a total height above lowest ground level of 130 feet, and has a full supply capacity of about 300,000,000 cubic feet.

From the above descriptions it will be seen that the Spanish system of maintaining reservoirs free from silt has great limitations, and is applicable only to high dams throwing the water back a comparatively short distance, and where a considerable perennial discharge is available. Examples of this kind are very rare under conditions obtaining in South Africa, Australia, and the United States, and the system is not likely to find application outside Spain.

Speaking generally, so far as the future is concerned, if all material inconvenience by sedimentation can be prevented for a period of eighty to one hundred years, the works will by that time have long since discharged their capital debt, and in practically all cases the raising of the full supply level by only ten feet will have the effect of increasing the original effective storage capacity by from sixty-six to one hundred per cent, a matter which can be effected at no great cost.

SILT IN CANALS

Turning now to the question of silt deposit in canals, we have here a problem which is more or less capable of solution. This particular branch of the subject falls into three distinct sections:

Firstly, the exclusion of as much sand and silt from the canal at the head works thereof as is possible.

Secondly, the removal by trapping and scouring devices of as

much as possible of the heavy deposit which cannot be kept out and accumulates in the head reaches immediately below the head regulator.

Thirdly, the design and grading of canals in such a manner that the finer silt, which is carried by the water in suspension beyond the region where it can be trapped or scoured, shall be maintained in suspension and thus carried into the watercourses and on to the land.

These three cases will be dealt with separately.

Design of Canal Head Works to Exclude Sand and Heavy Silt.—In no country has the problem of keeping sand and heavy silt out of the head reaches of canals been more serious than in Upper India, and the evolution of the design of the modern head works and the system of regulation adopted in that country gives an excellent idea of the conditions ordinarily obtaining and how these have to be met.

A modern head works in Upper India consists of the following essential portions:

(1) A weir across the river, which must be of sufficient height to force the required supply into the canal during low stages of the river. As Indian rivers are, generally speaking, subject to great variations of flow, these weirs are usually fitted with various patterns of drop shutters, which are raised during the low flow period and are dropped during the higher stages of the river or during floods.

(2) A regulator at the head of the canal, which consists of an arcade of openings controlled by sluice gates. The face of the regulator is generally parallel with the axis of the river, or, in other words, the axis of the canal is at right angles to the axis of the river.

(3) Under sluices.—These are placed at one or both ends of the weir joining the canal regulator or regulators. The object of the under sluices was originally to induce a strong current immediately in front of the regulator gates and thereby maintain a deep channel free from silt leading to the canal regulator.

(4) The divide groyne is a massive structure parallel to the axis of the river and to the canal regulator running upstream between the weir and the under sluices, and thus forms a defined approach channel to the regulator gates and to the under sluices.

This general design is common to practically all large modern head works, and is shown diagrammatically in the attached sketch, Fig. 29.

As recently as forty years ago Indian engineers when designing large head works did not pay much attention to the question of keeping silt out of the canals. The problem had not been studied scientifically, and the tendency was to look upon the silting up of canals, to a very considerable extent, as an unavoidable evil, which was to some extent overcome, in the first instance, by the costly method of annual silt clearances, or by

FIG. 29.—Diagrammatic sketch plan of typical Indian head works.

establishing large escapes through which the whole discharge in the canal could be passed back to the river in such a manner as to scour a considerable amount of silt from the canal from some distance above it.

Things proceeded in this haphazard and costly manner until the completion of the big Sirhind canal, in the Punjab, introduced a state of affairs which caused the engineers of the day the greatest anxiety, and in 1893 the main canal of this great system threatened to silt up entirely. From this time onwards much attention was devoted to the subject, and in 1898 a full official report of the greatest importance was drawn up and

published as a Punjab Irrigation Branch Paper No. 9, dealing with "The Deposit and Scour of Silt in the Main Line Sirhind canal and on Silt Experiments 1893 to 1898."

This report has been the starting point of a long series of improvements up to the present time. Dealing with the Sirhind canal problem the following remedies were applied:

(1) The escapes for scouring out silt from the main line were considerably enlarged.

(2) During the passage of high floods in the Sutlej, from which this canal takes off, the head regulator gates were closed completely so as to exclude water containing excessive quantities of coarse silt.

(3) A divide groyne of the type previously referred to was built, this important feature not having been included in the original design of the head works.

(4) The sill of the canal head regulator, which was originally two feet above the sill of the under sluices, was raised to seven feet above the canal bed level permanently, and in addition to this a movable sill was introduced, the crest of which could be raised to a maximum of three feet above the permanent sill.

The first of these remedial measures was certainly useful, but made no attempt to keep silt out of the canal. The closure of the canal during high floods was very effective, but it is an objectionable method in that it causes interruption in the supply and seriously interferes with the whole system of irrigation.

The divide groyne and the raised sill showed very marked results, provided that the under sluices were more or less closed, so that a comparatively still pond of water was created in front of the head regulator. It was found, however, that if the under-sluices were kept open the high raised sill had little effect in keeping out silt. As a result of the experience obtained with these improvements the "still pond" system was introduced in the case of several large head works. The under-sluice gates under this system are kept completely closed when the canal is open, so that a large pool of slow moving water is created, and the greater part of the heavy silt is deposited in this pool, instead of being carried into the canal, the water being drawn through the canal regulator over the raised sill. It stands to reason, however, that this process is not a continuous one. Heavy deposits of silt are formed in the pool, and unless these are scoured out by opening the under sluices from time to time

heavy silt would be drawn into the canal. This involves further interruptions in the continuity of supply.

In the first instance regulator sills were comparatively low. Later on, in some cases, the number of head regulator openings was increased, the sills were raised considerably so that the supply should be maintained by skimming the top water from the pool, and at the same time the under-sluice gates were kept open to a sufficient extent to maintain a scour through the pool and thus prevent an excess quantity of silt deposit forming in front of the regulator. This system is effective, but requires considerable care and judgment in working, and even so, under certain adverse conditions, it still becomes necessary to close the canal for a short period of time and open the under sluices completely so as to obtain a complete cleansing of the pool.

By the above-described means it has been possible to obviate silt troubles in the canal main lines to a very large extent. In the case of the great Indian canals it must be always borne in mind when dealing with silt problems that the extent to which the water is charged with silt varies enormously at different times of the year. For about six months the water in the rivers is entirely free from silt, or nearly so, whereas in the remaining six months the silt charge is very heavy. It has been found in consequence that a certain amount of silt deposit during the months when dirty water is flowing in the river is of no consequence, as during the periods of clean water flow this deposit is gradually picked up from the bed of the main canal and carried into the distributary system and then on to the land. So long as such a balance is maintained no harm results from silt deposit, provided the volume thereof is not so great as to materially interfere with the discharging capacity of the canal.

A valuable discussion of this particular silt problem is contained in Punjab Irrigation Branch Paper, No. 25, by Mr. F. V. Elsden, M.Inst.C.E., "Irrigation Canal Headworks, 1922," in which, apart from an historical account of past experiences and remedial measures adopted to deal with the trouble, Mr. Elsden makes some valuable suggestions. An abstract of this paper, which is not readily accessible, is given in "The Engineer" of November 10, 1922. Elsden's critical analysis and suggestions will be briefly referred to.

Water flowing in a stream carries silt in suspension owing to vertical eddies. These may perhaps be primarily caused by the roughness of the bed. These eddies may arise not only

from irregularities in the bed of the stream, but also from anything which causes an increased rate of change of velocity in a vertical plane.

This is an explanation of the state of affairs represented by Kennedy's formula $V_o = cd^m$ which was referred to earlier in the chapter, and which will be more fully dealt with later on.

In the design of all works for the conveyance of water the aim should be to obtain as steady a condition of flow as possible. This does not imply that the discharge should necessarily be uniform. Anything which causes unsteady flow gives rise to eddies which will disturb the silt regime of the channel and will thus bring more silt and coarser silt into the upper layers of the stream. It is therefore of primary importance to avoid all obstructions, sudden changes of cross section, and particularly any irregularities or obstructions the effect of which may extend to the lower layers of the stream flow.

To reduce the entry of silt into a canal to a minimum the following conditions must be complied with:

(1) Top water only must be admitted to the canal.
(2) The velocity of flow past the regulator must be as low as possible.
(3) Unsteady flow must be avoided.
(4) The bed of the under-sluice flume, or channel, between the groyne and the regulator must be as smooth as possible.

As regards condition (3), the usual design of head works is unsatisfactory. The regulator piers project beyond the gates and are provided with cutwaters, and they extend below the water surface down to sill level. Owing to the sudden change in the direction of flow of the water on entering the regulator heavy swirls are caused at the pier noses, and silt is stirred up from the bed and carried into the canal.

With regard to condition (1)—that top water only should be admitted to the canal—Elsden demonstrates that in most existing cases the works are constructed in such a manner that this condition cannot possibly be complied with when the regulation is such that water is flowing through the under sluices at the same time as it is flowing into the canal, an interesting experiment having been made in the case of one of the great head works, where 9,000 cusecs were being taken into the canal and 15,000 cusecs were passing through the under sluices, which amply corroborates his contention.

It is clear that in most cases a very large proportion of the water admitted into the canal over the raised sill is bottom water. This does not necessarily imply that silt from the lower layers of the stream will enter the canal. It is possible that all the silt charge in the lower layers does not rise with the water, and it is evident that the rolling sand at the bottom is certainly excluded.

There is ground for assuming that only a part of the bottom silt can be thus simply excluded from the water which enters the regulator. The direction of motion of the lower layers of water as they flow to the regulator will be obliquely upwards, and the velocity of this ascending water has a vertically upwards component which will carry to the upper layers silt which would not otherwise rise to that height.

Again in the dead area immediately adjoining the raised sill of the regulator there is no motion transverse to the main stream in the under-sluice approach channel, whilst water outside this area has a velocity towards the regulator superimposed on its motion in common with the main stream, with the result that there must be very considerable eddy motion set up, which will likewise tend to carry heavy silt in the top layers of water passing into the canal.

Either of these causes of upward movement of silt may by itself be of relatively small importance, but the two operating together and superimposed on the vertical eddies normally present in the stream may be considerable, and is probably sufficient to account for the fact that much harmful silt does actually get into canals in spite of the raised sills provided.

Elsden considers that it is clear that in a canal regulator of the usual type with its face at right angles to the under sluices a raised sill will only be completely efficacious for silt exclusion up to a certain maximum discharge, which will be a function of :

(*a*) the greatest depth from which water can safely be drawn ;

(*b*) the permissible velocity in the under-sluice flume ;

(*c*) the shape of the stream line curve of flow approaching the regulator ;

and, as a condition, it must further be laid down that the ratio of canal discharge to under-sluice flume discharge must not exceed a certain value, depending on the dimension (*a*) which must be judged from experience but should probably never exceed one-third of the depth of the under-sluice flume, in which

case the canal discharge should not exceed one-third of the discharge entering the under-sluice flume.

The permissible velocity in the under-sluice flume must also be judged mainly from experience, but may probably be put at a maximum of five feet per second in most cases.

The factor (*c*) is indefinite, and will be subject to variation from many causes, such as the shape of the regulator sill, the velocity of approach to the regulator, i.e. the velocity transverse to the axis of the under-sluice flume and the ratio of the depth of water over the regulator sill to depth in under-sluice flume. It would perhaps be possible to determine the shape of the curve in some typical cases, but in the main we must, at present, rely principally on judgment.

What is required for canals of any but the most modest dimensions is some form of head works which, while permitting only a moderate velocity in the under-sluice flume, shall ensure the arrival at the regulator of sufficient top water to supply the whole volume of discharge required for the canal, and this can evidently best be secured by aligning the face of the regulator along the plane which receives a maximum volume of top water per foot run. This plane is obviously the plane perpendicular to the direction of flow in the under-sluice flume, and it therefore becomes apparent that the best position of the regulator is such that the face is at right angles to the under-sluice flume, instead of being nearly parallel to it, as has been customary up to this time. The regulator must also have a raised sill, the level of which shall be the lowest level from which water can be drawn without admitting water which contains an excessive silt charge, and adequate means must be provided for the egress of the bottom water, which is not wanted in the canal, from the under-sluice flume.

All this points clearly to the necessity for the canal head regulator being superimposed on the under sluices, the remaining features of the head works, in general arrangement, being left much as they are at present designed, and the regulator sill being, say, one-third of the depth of flow below normal water surface level in the under-sluice flume. In this way the top three or four feet of water flowing down the under sluice flume will be skimmed off and passed into the canal, while the water in the remainder of the depth will be passed, with its silt, through the under sluices, and neither bottom water nor bottom silt will get into the canal.

Such a design naturally requires that a flow of surplus water be maintained in the under-sluice flume, and it might be objected that there may not always be water to spare for this purpose. This objection, however, has no real weight, for when the supply of water is restricted it is unlikely that it will be carrying a heavy burden of silt, and in such circumstances, there will be no harm in closing the under sluices completely; for though this will entail the admission of bottom as well as top water, this bottom water will carry with it no silt which can do any harm.

By this expedient of superimposing the canal head regulator on the under sluices the supply of true top water to the canal will be maintained in a natural way, and there will be no turbulent flow due to sudden changes of direction.

In addition to this essential change in the position of the canal head regulator Elsden suggests what he terms a skimmer gate of unusual design. The regulator sluice gate consists of two distinct parts. The sluice gate proper is of the usual design and works in vertical grooves on the face of the regulator. The skimmer gate is hinged at its lower end just below the sill of the regulator sluice opening and immediately upstream thereof, and can be rotated upwards in order to adjust, to varying water levels, the height at which water is taken into the canal. This arrangement makes it possible to alter the level of the regulator sill whilst preserving a sharp cutwater against the current, thus reducing to a minimum eddies caused by the presence of the gate, and giving a gradual entry to the under-sluice culverts.

The essential features of Elsden's design are shown in the accompanying sketch, Fig. 30.

Elsden deals essentially with very large head works, but the principles involved are very much the same when we are dealing with canals of very much smaller dimensions; and in the case of canals fed by dirty flood water of intermittent rivers in South Africa there is no reason why many of the principles enunciated above, and even Elsden's proposed layout of a head works, should not be adopted with advantage. The usual features of the larger South African head works are a solid diversion weir across the parent river, which is generally more than ten feet in height. Weirs with a height of twenty feet and upwards are common. It is rare, however, to find shutters or under sluices on or in the weir section. It is therefore usual to place the canal regulator sill at a sufficient depth below the weir crest to ensure the full discharge of water into the canal when the river discharge

is just adequate to provide it and no water is discharged over the weir.

Under these conditions the exclusion of silt from the main canal is clearly impossible, and the design of these works deliberately encourages the influx into the canal, not only of heavy silt, but of rolling sand, and even gravel and boulders. Whenever the weir is high enough Elsden's superimposed design could be adopted with great advantage.

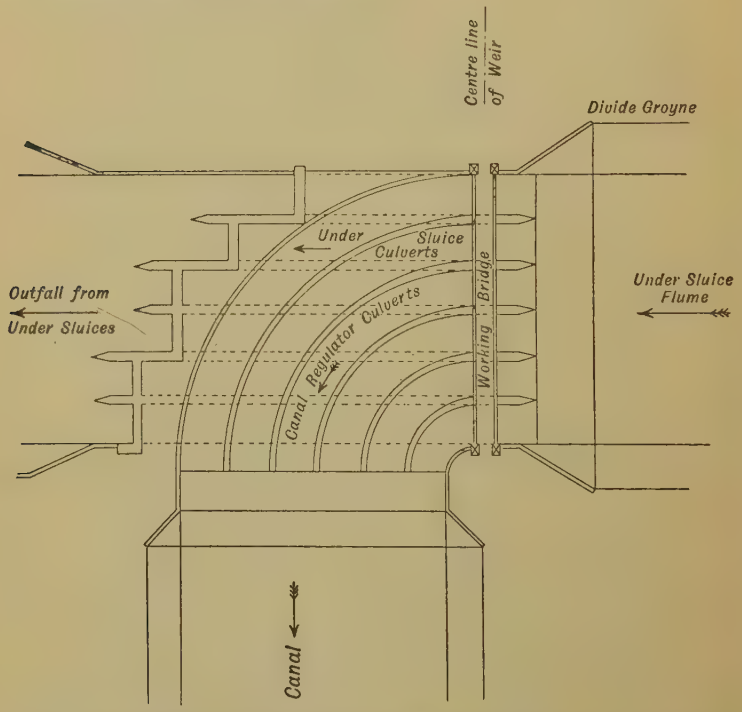


FIG. 30.—Sketch plan of Elsden's proposed design for head works, with canal regulator superimposed on under sluices.

It is essential, however, that when this is done a properly constructed straight approach flume should be provided having smooth sides and a smooth bed. Speaking generally, however, silt and sand cannot be excluded from canals of this kind, and provision must be made for trapping as much of it as possible close to the head regulator and for scouring out the deposit as often as required.

Sand traps.—In cases where heavy and gritty deposits are apt to be carried far down the canals, as frequently happens when the parent stream is torrential in character, traps are introduced close to the head regulator which, by artificially checking the velocity of the water flowing into the canal, cause the deposition of all the heavy solids. Such traps are particularly necessary in the case of power canals, as even the coarser suspended solids have an extremely erosive effect on the buckets or vanes of pelton wheels or turbines.

Some very efficient designs of this kind have been evolved in Switzerland and France, the most effective being designed and patented by a Swiss engineer, M. Henri Dufour, of Bâle, the device being known as a *dessableur*. These devices vary considerably in design according to the nature of the material to be trapped, but they consist essentially of an elongated concrete stilling chamber containing baffles or other devices for lowering the velocity sufficiently to cause all heavy material to drop to the bottom, the cleaned water being skimmed or taken from the upper layers. The traps are generally made in duplicate, so that the deposit in one section can be scoured out whilst the other is working.

Some typical traps of this kind are described in the " Annales Suisse d'Hydrographie," Vol. II, 1916, published officially in Berne in a report on " Le Charriage des Alluvions " by Dr. Leon W. Collet, and also in an article by Henri Dufour in the " Bulletin Technique de la Suisse Romande," Nos. 2 and 3 of 1922. A number of American examples are given in Etcheverry's book on " Irrigation Practice and Engineering."

Scour Sluices.—Where sand traps of special types are not installed scour sluices should be provided. These should be located near to the head regulator, and preferably more than one should be provided.

To achieve the best results the head reach should be graded at a sufficiently flat slope to ensure that all heavy silt, sand, etc., will be deposited in the reaches above the scour sluices.

The length of the reach to be scoured by a single sluice should be adjusted to the size of the canal. In the case of comparatively small canals, such as are usual in South Africa, the silt trap reach should not exceed 500 feet in length. If the reaches are made too long it becomes difficult to scour them out rapidly and without undue waste of water. It is often the case that the sites chosen for the scouring sluices are determined

by rocky outcrops in the banks, as, if these do not exist, expensive structures are required to lead the water from the scour sluices back to the river.

The scour sluices should be placed at the end of the trapping reaches with their sill levels eighteen inches or more below the normal bed level. The longer the reach to be scoured the lower should be the sill of the sluice. Immediately below the scour sluice a regulator must be built in the canal, so that the latter can be completely shut off and all the water turned into the scour sluice. The regulator must be provided with a substantial sill at normal bed level, so that the grading of the canal shall not be interfered with when scouring takes place.

The bed of the canal in front of the scour sluice may either be constructed as a masonry or concrete well or pit, so that the canal bed above it is maintained after scouring operations are complete at the normal grading; or the bed of the canal may be allowed to scour down to a steeper slope determined by the sill level of the scour sluice. This latter arrangement is better if the reach is fairly long, as it permits of more effective scouring and gives more capacity for silt deposit.

In the case of very dirty rivers it is advisable, if possible, to provide several trapping reaches and scour sluices, placed one below the other.

Below the lowest scour sluice the canal must be made more or less self cleansing in the manner briefly described in the following paragraphs.

SILT PREVENTION IN CANAL SYSTEMS

Coming now to the question of the prevention of silting in canals apart from the deposits in the head reaches above dealt with, Kennedy, as was stated at the beginning of this chapter, showed that the velocity of water flowing in a canal in permanent regime is independent of all factors excepting the depth. As the results of many experiments made in a section of the Bari Doab canal system, in the Punjab, Kennedy found that the critical non-silting and non-scouring velocity was

$$V_o = 0{\cdot}84 d^{0{\cdot}64}.$$

When this theory was first put forward by Kennedy in 1895 (Paper 2826, Vol. CXIX of Proc. Inst. C.E.) he claimed that in two channels of the same depth and the same mean velocity, but one being much wider and of greater discharge than the

other, both channels will have the same silt-carrying power because the number of small upward eddies which alone support the silt are proportioned to the width, and therefore also to the volume of water.

The effect of increasing the depth of either channel, while maintaining the same mean velocity, would be to reduce the power of the eddies to keep silt in suspension.

The first practical application of this theory was to design channels with wide beds and small depths of water, but it was soon discovered that such extreme sections were very unsatisfactory. It was subsequently demonstrated that V_o is not a function of the depth d alone. The ratio of breadth to depth $\frac{b}{d}$ must increase with the discharge, and though a ratio of $\frac{b}{d} = 12$ may be consistent with good regime in a channel discharging 1,000 to 2,000 cusecs, this may not be the case with a channel discharging less than 100 cusecs.

This was later fully recognized by Kennedy himself, and his instructions for grading and designing irrigation channels (Punjab Irrigation Branch Paper No. X, 1904) contains the following:

The ratio of bed to depth will, of course, vary with the size of the channel. For small channels of 10 cusecs the maximum limit may be 3·5. For channels with discharges of 25 cusecs, 4·0; for 100 cusecs, 4·5; for 200 cusecs, 5; and so on.

Channels designed with larger ratios than these invariably gave trouble by excessive berm growth. As the result of many years' experience with channels designed on Kennedy's principles the Punjab Irrigation Department issued, in 1917, certain rules for the design of these channels, fixing the ratio $\frac{b}{d}$ for canals of all capacities ranging from 2 to 20,000 cusecs. Some of these ratios are given below.

Discharges not given above have intermediate values.

Kennedy published a set of discharge diagrams based upon Kutter's formula for a very full range of ordinary earthern channels which show the relation for each gradient specified between the bed width, depth of water, velocity, and discharge. In addition to this information there are plotted on the diagrams curves representing the critical or non-silting, non-scouring

velocity V_o, so that for any given gradient and discharge the section which will give theoretically a non-silting velocity can be found. All canal sections lying to the right of the V_o line will scour, whereas all those to the left of it will silt.

Discharge	Ratio of Bed Width to Depth
2	2·0
4	2·3
6	2·6
8	2·75
10	2·9
12	3·0
30	3·5
80	4·0
175	4·5
300	5·0

This particular critical velocity curve is, however, representative of a particular grade of silt which happened to be present in the channels in which the original experiments were made by Kennedy. If the silt to be dealt with is coarser or finer than the silt experimented with, the critical velocity will have to be increased or diminished. To meet this difficulty Kennedy located a series of fractional lines on either side of the V_o line, such as $0{\cdot}7V_o$, $0{\cdot}8V_o$, $0{\cdot}9V_o$ on the silting side, and $1{\cdot}05V_o$, $1{\cdot}1V_o$, etc., on the scouring side.

Once therefore it is possible to determine the value of the critical velocity for a particular grade of silt in terms of V_o it becomes possible to design the channels so as to be non-silting. On an irrigation furrow of considerable length with numerous off-takes the tendency is for the heavier particles of silt to be withdrawn from the water by off-takes in the upper reaches so that the silt in suspension becomes progressively finer as the water flows down the canal. Thus whilst the critical velocity at the head of a canal may be $1{\cdot}1V_o$, it may be only $0{\cdot}7V_o$ near the tail.

The effectiveness of the application of Kennedy's principles of design depends entirely upon the value adopted for the critical velocity at the head. Where we are dealing with muddy rivers, and adequate precautions have been taken to keep out, or scour out most of the heavy silt, it is obviously useless to attempt to

determine the critical velocity ratio on the basis of the amount of silt contained in the parent stream. The value to be adopted will be smaller than would be applicable to the crude water.

It is likewise necessary to remember that the silt content varies greatly from time to time and mean conditions must be taken as a basis. In South Africa conditions are very erratic and experience very limited. It is therefore not possible to prescribe definite values for the critical velocity ratio for different sources of supply.

In the Transvaal, where rivers are very free from silt, values from $0{\cdot}8V_0$ to V_0 would probably apply.

Where we have to deal with muddy flood water only, as is the case with the flood canals of the Cape province, the critical velocity immediately below the scour sluices in the canals should be assumed to be from $1{\cdot}2V_0$ to $1{\cdot}1V_0$.

With these flood irrigation furrows it is generally advisable to give them a grade and section which will ensure a slight scour. They are nearly always cut in fairly stiff soil, so that no serious harm will result. It is comparatively simple to rectify a fault in this direction. It must be borne in mind also that these canals invariably carry muddy water.

The latest edition of Kennedy's diagrams are calculated on the basis of Kutter's coefficient $N=0{\cdot}0225$. Such a low value is seldom obtainable in South African practice, and the value of N will generally be found to be 0·025 for canals in earth in fair order, and may be as much as 0·03 in rock cuttings, etc.

The effect of increasing N from 0·0225 to 0·025 is to decrease the velocity by approximately 10 per cent.

The effect of increasing N from 0·025 to 0·30 is to decrease the velocity by 17 per cent. Kennedy's diagrams are strictly applicable only to canals in permanent regime with silted berms having side slopes of one-half to one. Channels with other side slopes than this will carry greater or less discharge, as the case may be.

Buckley in the "Irrigation Pocket Book" (3rd Edition, 1920) gives a nomogram which enables Kennedy's critical velocity and critical velocity ratio, and also critical velocities for other selected constants, to be readily found.

CHAPTER VIII

DUTY OF WATER AND CANAL CAPACITIES

THE measure of efficiency of an irrigation system is the area irrigated per annum in relation to the quantity of water taken into the canal at its head. This relationship is expressed in various forms and is usually known as the "duty" of water. As the duty forms a basis of comparison between different irrigation systems in the same or in various countries, and as it is the foundation upon which every irrigation scheme is built up, it is essential that the term should be clearly defined, so that there may be no risk of applying duty figures in a wrong manner. Canal duties are calculated with various objects and are most commonly expressed as follows:—

(*a*) The area irrigable per annum in terms of the full supply capacity of the canal at the head. This is more properly termed the "full supply factor," and is expressed as the area irrigable annually in acres by each cusec of full supply capacity at the head of the canal.

(*b*) The area actually irrigable per annum in acres in terms of the mean discharge which has actually been taken into the canal for irrigation purposes. Any water discharged from the canal system through scouring sluices, or through escapes along the canal, or at the tail, must in this case be deducted from the amount entering the canal.

(*c*) The area actually irrigated per annum in terms of the discharge in cusecs delivered to the crops at the field boundaries. The difference between the duty calculated by this method and the previous one represents the canal losses between the head and the outlet into the fields. The duty calculated by this method is also a measure of the efficiency of the methods of

the irrigator and can only be controlled to a limited extent by the canal engineer.

(*d*) Each of the above duties may be expressed in terms of depth of water delivered on to each acre or the unit area of land irrigated.

Thus, for example, the field duty described in paragraph (*c*) above might be fifteen acre inches per annum. A few examples of the above-mentioned methods of expressing duty are annexed.

The duties expressed in terms of (*a*) and (*b*) above, whether in acres per cusec or in acre inches, are useful for general statistical purposes, and may be used as a basis for the design of new projects where these are large, losses more or less known and fairly constant, and agricultural conditions uniform. Such is the case in Upper India, for example, where a vast new project, irrigating, say, 1,000,000 acres may be designed on the basis of well-established full supply factors.

Care must be exercised when applying duty statistics, especially if these refer to conditions in other countries. In hot countries, such as India, the time taken to bring a given crop to maturity under irrigation is much shorter than in South Africa, and double and even triple cropping is as common in India as it is rare in South Africa.

Indian duties are therefore nearly always seasonal duties, and are given separately for the kharif or summer crops and for the winter or rabi crops.

As some summer crops, such as rice, require a very large amount of water, the land becomes so heavily charged with moisture that it is possible to grow a wheat or other cereal crop on the same land after the rice crop has been reaped without applying any further water, or, at the most, with one additional watering. Under such conditions abnormally high winter and annual duties are frequently obtained. Some Indian duties are given at the end of this chapter.

Under the variable conditions prevailing in South Africa this method is inadmissible excepting for rough first approximations. The only safe method is to build up the scheme from the field duties, and it is preferable to express these in terms of acre inches. Under agricultural conditions prevalent in South Africa the seasonal demand varies greatly, and it was emphasized in an earlier chapter that the critical irrigation period throughout the Union is during the spring and early summer months, August, September, and October. It may be taken as a universal

rule that during one of these months at least the whole of the area irrigated during the year will require a watering and a somewhat liberal one. Thus if the net annual irrigable area and the mean depth of water required for the heavy spring watering are known, the maximum demands likely to be made on the canal system are calculable. It is presumed in this case that a single watering during the month is sufficient. Should certain special crops, such as vegetables or seedlings, require more frequent irrigation an additional allowance must be made. Neglecting such special requirements, we are able to calculate the total volume of water which must be delivered on to the crops during a period of thirty days, expressed in acre feet. This will represent the maximum demand on the canal system and therefore determine the carrying capacity of the canal and distributing channels from the head downwards.

When designing a new irrigation project the detailed contour and soil surveys should provide accurate data regarding the gross area of commanded land which is culturable and physically and economically irrigable. This area constitutes the gross culturable and irrigable area. This is, however, in excess of the area which will be actually irrigable, as an appreciable percentage of the gross area will be occupied by roads, furrows, drainage lines, buildings, spaces required for farming operations, and other items.

The proportion of gross irrigable land so occupied and then rendered non-irrigable will vary considerably with the character of the scheme, and is usually greater in inverse proportion to the size of the individual holding. It will generally vary from ten to twenty per cent. The net irrigable area will therefore be some fraction p of the gross area A.

Theoretically the net area a is actually irrigable annually, but it is seldom that this is the case.

On a small scheme, or one comprising a number of very small holdings, it may be assumed that the entire net area will be irrigated during each year, but as the schemes grow and individual holdings become larger there is a growing percentage of the net area which for some reason or other is permanently out of cultivation. In some cases—in northern India, for example—the percentage of the net area which may be irrigated during each year is arbitrarily fixed by authority, in order to minimize the evils of excessive concentrated irrigation; but we are here dealing only with a certain voluntary exclusion of land

from irrigation for varying periods of time. Under South African conditions, and assuming full development of the irrigated area, the percentage of the net irrigable area which is likely to be actually irrigated at any particular period will vary from eighty-five per cent in the case of large schemes to ninety-five per cent in the case of small schemes. If this percentage is expressed by the coefficient K, then the maximum area for which water must be provided during the month of greatest demand will be $a = K \times p \times A$ acres.

The next thing to consider is the actual time within which this area a must be irrigated. On all large schemes it is essential that water should be taken into the canal at the head continuously, and for reasons of economy in the use of water it is most desirable that so long as the demand for water exists and water is available the flow in the canal should be continuous. The frequent closing and opening of a canal makes it impossible to maintain it in good regime, greatly increases transit losses, and lowers the duty. In all the old irrigating countries of the world irrigation is practised day and night, and in hot dry weather night irrigation is, from an agricultural point of view, better than if carried on during the heat of the day.

On very small schemes in South Africa farmers are very averse to night irrigation, and frequently cease operations on Sundays also.

Even in comparatively small schemes, when the canal exceeds ten miles in length, it is generally possible to curtail night irrigation, if required to do so, by working the canal in sections.

In the case of pumping schemes, it is usual to work the pumping plant for a portion of each day only. The periods vary from eight to twenty hours per day.

If h represents the number of hours per days and n the number of days during the month during which irrigation is practised, the time T in seconds per month during which water must be delivered will be $3600\ h \times n$.

In most cases, during the month of maximum demand, $T = 3600 \times 24 \times 30 = 2{,}592{,}000$ seconds. Now the water to be delivered on to the crops will be expressed in depth in inches or preferably feet. Thus a three inch watering will be expressed by $\triangle = 0{\cdot}25$ feet. If the net irrigable area is given in acres the discharge in cusecs required to irrigate the whole area during the critical period will be

$$D = \frac{a \times 43560}{3600 \times h \times n}.$$

If $n = 30$ days, $D = 0{\cdot}403\,\frac{a}{h}$.

If $h = 24$ hours, $D = 0{\cdot}0168a$.

Thus, for example, if $A = 1500$ acres, $p = 0{\cdot}85$ and $k = 0{\cdot}9$.

Then $a = k \times p \times A = 1147{\cdot}5$ acres;

and if $\triangle = 0{\cdot}33$ feet,

$D = 0{\cdot}0168 \times 1147{\cdot}5 \times 0{\cdot}33 = 6{\cdot}1$ cusecs.

By the above method the maximum discharge required to be delivered on to the land during the most critical month of the year can be calculated for a continuously flowing or perennial canal. In the case of flood or intermittently flowing canals the method of calculating the water requirements is different. In these cases we are not particularly concerned with critical periods. The water supply being of an erratic and intermittent character the irrigation position may be described as always critical, and whenever water is available the entire net irrigable area must receive a heavy flooding.

The time during which this irrigation must be accomplished is beyond control in these cases, and is determined by the number of days during which sufficient water is available in the river. When designing a project of this nature it is necessary to ascertain as nearly as possible the average number of days during which the river flows on each occasion when a freshet or flood comes down. During these intermittent periods of flow the discharge ranges from very large amounts during the height of the flood to a mere trickle at the end of the period. If, for example, the flow period lasts seven days, the discharge in the river may be sufficient during the first three days to fill the canal. During the remaining four days the supply in the canal gradually diminishes, until it reaches a stage when it becomes too small to be distributed. According to the nature of the freshets, therefore, the net water requirements should be based on the supposition that the canal is running one-half, two-thirds, or three-quarters full throughout the average period of flow.

If T is the average period of a freshet in days, and the canal is assumed to be running two-thirds full throughout that period, then the net water requirements in terms of discharge in cusecs will be

$$\frac{a \times 43560 \times 3}{T \times 86400 \times 2} \times \triangle,$$

where $\triangle$ is the depth of water to be delivered on to the land. In cases of this kind $\triangle$ should not be less than six inches.

Having thus found the maximum discharge which must be actually delivered on to the land, the next step is to calculate the absorption and other losses throughout the canal system, and then find the actual capacity of the canals from the head downwards. For this purpose the quantity of water to be delivered by each section of the canal must be computed. The absorption and other losses must be calculated reach by reach, according to the nature of the ground traversed by the canal itself. The capacities should be built up from the tail, working upstream. As the losses are computed in terms of cusecs per 1,000,000 square feet of wetted area as described in the chapter on losses, it will be necessary, in the first instance, to assume a mean cross section for each reach which will pass the required discharge, plus, say, ten per cent for losses. From this mean section the actual losses can be computed and if necessary the mean cross section amended.

The discharge at the head is made up of the total discharge to be delivered on to the land, plus all the computed losses in canals and the distributaries below.

On extensive canal systems in earth carrying silt-laden water, if the gradients and sections are chosen in accordance with Kennedy principles, the capacities are based upon a stable regime with silted side berms having ½ to 1 side slopes. The canals will, however, be made initially with flatter slopes, viz., 1 to 1, or 1½ to 1, so that until the silt berms have formed the canals will have a capacity somewhat in excess of requirements.

If ample supplies of water are available nothing further than the above would be required, but this is seldom the case. The total volume of water available annually is nearly always less than is required to irrigate the land commanded by a scheme. We have therefore, in the first instance, to determine what area of land can be efficiently irrigated with the supplies available in all but exceptionally bad years. A crop must not only receive water at certain critical times, it must obtain a certain minimum amount of water distributed throughout the growing season, either in the form of rain or by means of irrigation.

Irrigation water has to make up deficiencies in the amount of seasonal distribution of rainfall.

No arbitrary rules can be laid down as to what depth of water should be delivered on to the land during the year. This depth varies with the crop grown, with the climate, and also with the

soil. A sandy loam will produce crops with the application of much less moisture than is the case with a clay soil.

As regards climate, in many hot and dry regions the rainfall must frequently be ignored altogether; on the other hand, in parts of the northern and eastern Transvaal it is only necessary to provide for one, or at the most two, irrigations early in the growing season, and the rainfall will suffice during the remainder of the season.

In the dry parts of the Union, where the effect of the rainfall may be ignored, and assuming that the soil is a medium to sandy loam, the total depth of water which must be delivered on to the land irrigated varies from twelve to eighteen inches if the supply is available when required. In the case of intermittent or flood irrigation canals a larger allowance has to be made, as owing to the uncertainty of supply very heavy waterings are given, so as to make their effect as lasting as possible.

Flood irrigation schemes invariably work with a low duty, whereas on an efficient perennial canal with skilful and experienced cultivators the duty is sometimes extraordinarily high.

We will here confine our attention to perennial systems, as these are the more important and are represented by most schemes served by storage reservoirs.

There is a great difference between the amount of water theoretically necessary for a crop and the quantity required to produce a good crop under ordinary working conditions. Very much depends upon the standard of husbandry and the skill and experience of the irrigator. If the irrigator is inexperienced or lazy, or if he endeavours to irrigate more land than he can properly control, excessive use of water on the lands and waste in other directions are inevitable.

No lesson appears more difficult to learn than the fact that excessive irrigation produces poor crops, and ruins the land by waterlogging it and bringing injurious brak salts to the surface. Elaborate field experiments carried out on a large scale in India, the United States, and elsewhere have shown clearly that a particular number of waterings of certain depth will produce the best outturn of crop. Any excess as regards number or depth will either produce only an insignificant additional yield or no increase at all, and may even give rise to a diminished output. The economical amount of water to be given to a crop is not necessarily that which will give the absolute maximum yield, but the minimum which will give a good and economically

profitable yield. Thus the elaborate wheat duty experiments in the Punjab clearly showed that in the case of land which had been under irrigation and cultivation for some years the economic limit was attained with three moderate waterings each of about three inches in depth. Greater depth of water or more frequent floodings produced small increases in yield, quite out of proportion to the value of the additional quantity of water used.

Striking evidence of the effect of using an excess of water is given in the three tables appended relating to wheat, maize, and lucerne. The data were obtained at the Utah Agricultural Experimental Station, and are taken from Widstoe's book on "The Principles of Irrigation Practice." It is important to note also that in a recent case which came before the Water Court in South Africa it was held to constitute unreasonable use of water if a riparian owner exceeded this economic limit. The court stated that if a given amount of water per acre will produce a good crop an irrigator is not entitled to use a large additional amount of water to secure an increase of only five or ten per cent.

Under average existing conditions, in the drier parts of the Union, twelve to fifteen inches of water per annum is ample for the requirements of most crops. Where lucerne is grown exclusively the annual allowance may be increased to eighteen inches, which should secure six full cuttings of this crop during the year.

There are, however, numerous directions in which it is possible to economize in the use of water, whereby the general standard of agriculture can be improved and a greater area placed under irrigation with the water available.

TABLE XXXI

Yields of Wheat with Varying Quantities of Irrigation Water.
(Utah Experimental Station.)

Inches of Irrigation Water Applied	Bushels of Grain to the Acre	Pounds of Straw to the Acre	Pounds of Straw for each Bushel of Grain	Bushels of Wheat for each Inch of Water
5·0	37·81	2,986	79	7·56
7·5	41·54	2,301	75	6·39
10·0	43·53	3,452	79	4·35
15·0	45·71	3,954	87	3·05
25·0	46·46	4,311	93	1·86
35·0	48·55	4,755	98	1·39
50·0	49·38	5,332	108	0·99

TABLE XXXI—*Continued.*

Yields of Maize with Varying Quantities of Irrigation Water.

(Utah Experimental Station.)

Inches of Irrigation Water Applied	Bushels of Grain to the Acre	Pounds of Stover to the Acre	Pounds of Stover to each Bushel of Maize	Bushels of grain for each Inch of Water
7·5	79·14	7,189	91	6·07
10·0	89·52	6,007	67	5·80
15·0	93·93	8,279	88	4·57
20·0	91·58	8,692	95	3·59
25·0	99·16	9,492	96	3·25
30·0	97·12	10,390	107	2·73
55·0	96·78	10,258	106	1·43

Yields of Cured Lucerne (Alfalfa) Hay with Varying Quantities of Irrigation Water.

(Experiments by S. Fortier, Montana.)

Inches of Irrigation Water Supplied.	Pounds per Acre	Pounds per Inch of Water
6	9,220	1,537
12	8,840	737
18	7,500	416
24	12,700	529
30	14,400	480
36	15,360	426

TABLE XXXII

Results of Indian Experiments showing Relation of Yield to Irrigation Water Used.

Number of Waterings	1	2					3				
Upper Bari Doab Canal, Jaura. Old land. 1907 *to* 1908.											
Total depth of water used (ft.)	0·25	0·25	0·38	0·50	0·63	0·76	0·38	0·57	0·75	0·94	1·14
Total rainfall in feet	0·31	0·31	0·31	0·31	0·31	0·31	0·31	0·31	0·31	0·31	0·31
Total water on crop (ft.) ..	0·56	0·56	0·69	0·81	0·94	1·07	0·69	0·88	1·06	1·25	1·45
Grain yield in maunds per acre	6·19	6·87	8·35	7·51	10·01	8·58	11·38	11·05	9·96	9·90	8·16
Straw yield in maunds per acre	5·85	8·04	8·87	8·87	11·83	10·92	16·57	13·01	19·06	14·98	11·83
Lyallpur, Lower Chenab Canal. Old land. 1907 *to* 1908.											
Total depth of water used (ft.)	0·237	0·234	0·364	0·505	0·617	0·719	0·364	0·541	0·709	0·900	1·076
Total rainfall (ft.)	0·124	0·124	0·124	0·124	0·124	0·124	0·124	0·124	0·124	0·124	0·124
Total water on crop (ft.) ..	0·361	0·358	0·488	0·628	0·741	0·843	0·488	0·665	0·833	1·024	1·200
Grain yield in maunds per acre	4·66	10·56	9·60	10·79	19·74	17·97	16·61	10·14	15·46	23·01	19·93
Straw yield in maunds per acre	9·18	19·49	15·65	18·20	36·29	39·02	28·96	16·03	24·62	41·77	34·80
Upper Bari Doab Canal, Pattoki. Old land. 1907 *to* 1908.											
Total depth of water used (ft.)	0·251	0·376	0·438	0·500	0·625	0·503	0·503	0·628	0·755	0·878	0·998
Total rainfall (ft.)	0·101	0·101	0·101	0·101	0·101	0·101	0·101	0·101	0·101	0·101	0·101
Total water on crop (ft.) ..	0·352	0·477	0·539	0·601	0·662	0·726	0·604	0·729	0·856	0·979	1·099
Grain yield in maunds per acre	4·5	7·3	7·5	8·0	4·7	14·0	12·0	11·0	13·5	14·5	12·3
Straw yield in maunds per acre	8·2	13·5	9·5	13·5	11·1	14·5	25·1	26·5	25·5	26·1	23·0
Lower Jhelum Canal.											
Total depth of water used (ft.)	0·258	0·233	0·362	0·596	0·627	0·772	0·365	0·560	0·768	0·971	1·193
Total rainfall (ft.)	0·212	0·212	0·212	0·212	0·212	0·212	0·212	0·212	0·212	0·212	0·212
Total water on crop (ft.) ..	0·470	0·445	0·574	0·808	0·839	0·984	0·587	0·772	0·980	1·183	1·405
Grain yield in maunds per acre	2·41	4·77	7·54	8·99	10·00	10·62	7·33	8·00	10·70	11·37	12·08
Straw yield in maunds per acre	5·92	9·04	12·15	12·85	14·58	16·04	9·79	11·59	20·60	20·0	19·4

TABLE XXXII (*continued*)

Number of Waterings	4					5				
Upper Bari Doab Canal, Jaura. Old land. 1907 *to* 1908.										
Total depth of water used (ft.) ..	0·49	0·76	1·00	1·25	1·51	0·60	0·93	1·21	1·55	1·90
Total rainfall in feet	0·31	0·31	0·31	0·31	0·31	0·31	0·31	0·31	0·31	0·31
Total water on crop (ft.)	0·80	1·07	1·31	1·56	1·82	0·91	1·24	1·52	1·86	2·21
Grain yield in maunds per acre ..	12·56	13·39	12·15	12·35	6·27	9·45	9·17	11·09	6·14	6·27
Straw yield in maunds per acre ..	16·48	21·55	18·46	18·20	9·34	13·60	14·04	15·01	9·13	11·70
Lyallpur, Lower Chenab Canal. Old land. 1907 *to* 1908.										
Total depth of water used (ft.) ..	0·469	0·709	0·942	1·166	1·395	0·531	0·797	1·043	1·101	1·320
Total rainfall (ft.)	0·124	0·124	0·124	0·124	0·124	0·124	0·124	0·124	0·124	0·124
Total water on crop (ft.)	0·593	0·833	1·066	1·290	1·519	0·655	0·921	1·167	1·225	1·444
Grain yield in maunds per acre ..	10·48	12·12	12·10	15·27	16·48	13·09	14·99	17·05	21·59	24·92
Straw yield in maunds per acre	15·55	18·43	17·63	27·46	27·42	17·24	23·32	26·63	41·43	43·94
Upper Bari Doab Canal, Pattoki. Old land. 1907 *to* 1908.										
Total depth of water used (ft.) ..	0·504	0·684	0·865	1·061	1·245	0·630	0·872	1·127	1·375	1·625
Total rainfall (ft.)	0·101	0·101	0·101	0·101	0·101	0·101	0·101	0·101	0·101	0·101
Total water on crop (ft.)	0·605	0·785	0·966	1·162	1·346	0·731	0·873	1·228	1·476	1·722
Grain yield in maunds per acre ..	9·4	13·7	16·5	19·9	20·8	10·5	14·5	18·7	19·8	19·5
Straw yield in maunds per acre ..	21·5	26·5	34·0	40·5	41·9	14·5	29·0	31·6	39·0	49·0
Lower Jhelum Canal.										
Total depth of water used (ft.) ..	0·483	0·742	1·011	1·207	1·562	0·635	0·963	1·336	1·645	2·180
Total rainfall (ft.)	0·212	0·212	0·212	0·212	0·212	0·212	0·212	0·212	0·212	0·212
Total water on crop (ft.)	0·695	1·954	1·233	1·482	1·784	0·847	1·175	1·548	1·857	2·392
Grain yield in maunds per acre ..	9·65	9·73	10·70	9·65	12·59	12·03	12·35	12·76	12·33	14·11
Straw yield in maunds per acre ..	20·2	20·03	23·69	20·65	19·91	20·19	21·06	28·14	24·5	24·2

TABLE XXXIII

Statement showing some Typical Duties obtained on Various Canal Systems in the Punjab (India) in the year 1919–1920.

Name of Canal System	Name of Distributary	Area Irrigated in Acres			Duty in acres per Cubic Foot of Average Discharge		Depth of Water Used on Area Irrigated including Losses in Distributary		Remarks
		Kharif (Summer)	Rabi (Winter)	Total	Kharif	Rabi	Kharif	Rabi	
		Acres	Acres	Acres	Acres	Acres	Feet	Feet	
Bari Doab Canal	Fatehnangal	6,096	5,382	11,478	139	598	2·6	0·6	Typical rice distributaries. The entire winter crop of cereals grown without further irrigation
,, ,,	Gulpur	777	602	1,379	86	602	4·2	0·6	Perennial distributary
,, ,,	Thamman	35,011	36,198	71,209	161	241	2·2	1·4	
Lower Chenab	Rakh Branch—average of 22 distributaries	91,753	165,276	257,029	123	261	2·60	1·3	
,, ,,	Fatch	180	311	491	180	311	1·8	1·1	One of the smallest distributaries from Rakh Branch and hence gives highest duty

TABLE XXXIII (*continued*)

Name of Canal System	Name of Distributary	Area Irrigated in Acres			Duty in Acres per Cubic Foot of Average Discharge		Depth of Water Used on Area Irrigated including Losses in Distributary		Remarks
		Kharif (Summer)	Rabi (Winter)	Total	Kharif	Rabi	Kharif	Rabi	
		Acres	Acres	Acres	Acres	Acres	Feet	Feet	
Lower Chenab	Bath	889	1,968	2,857	127	328	2·60	1·02	Small, medium and large distributaries from Upper Gugera Branch
,, ,,	Gajiana	8,874	18,070	26,944	111	238	3·00	1·41	
,, ,,	Sharkpur	25,383	47,287	76,670	141	242	2·34	1·39	
,, ,,	Nawab	1,267	2,145	3,412	181	306	1·80	1·10	Small and large distributaries from Lower Jhang Branch
,, ,,	Dhaular	26,436	52,570	79,006	107	219	3·10	1·50	
Lower Jhelum	Southern Branch average of 18 distributaries	106,112	198,786	304,898	109	209	3·0	1·7	
Sirhind	Bhatinda Branch average of 19 distributaries	147,970	224,075	372,045	137	204	2·4	1·8	
Western Jemna Canal	Whole system *	424,436	430,546	854,982	122	169	2·59	2·11	Most of system is very old
Sirhind Canal	,, ,, *	399,353	599,204	998,562	132	228	2·50	1·6	Native State irrigation excluded
Upper Bari Doab	,, ,, *	639,019	613,564	1,252,583	122	214	—	—	
Lower Chenab	,, ,, *	851,161	1,604,064	2,455,225	95	208	—	—	Largest Canal in India

* In these cases duties are calculated on basis of average discharge entering canal at the head, and these include all main line and Branch losses.

CHAPTER IX

DISTRIBUTION OF WATER ON TO THE IRRIGABLE LAND

DISTRIBUTION SYSTEM

IRRIGATION engineering does not stop at designing and constructing a main canal system. The success or otherwise of an irrigation scheme depends in many cases upon the skill with which the reticulation system is planned, i.e. the system of laterals or watercourses which carry the water to each individual irrigation plot.

Efficient and economical irrigation rests to a large extent with the farmer, and principally on the manner in which he lays out the land to be irrigated.

A cardinal principle in all irrigation practice is that no irrigation system can be successful without a complementary drainage system. Without the latter the land will very quickly deteriorate, and may even become useless, owing to the concentration of injurious brak salts near the surface of the ground, or owing to waterlogging, when the natural subsoil level is not far from the surface.

When planning a distribution system the first thing to do is to mark on a fairly large scale contoured plan of the irrigable land all the natural drainage lines which should show up as a comprehensive system. Main, secondary, and minor drainage lines should be clearly indicated by lines of different thickness. Similarly, main, secondary, and minor ridges should likewise be shown. Whether large blocks are to be irrigated or small individual holdings, the irrigable area should be divided up into comparatively small areas, each of which will be served by a separate watercourse, which may either take off from the main distributary or small canal direct, or from a branch distributary.

It is most desirable to limit as much as possible the number

of outlets or " turnouts " on a small public canal or large distributary. If there are too many of them the regime of the canal is difficult to maintain, especially if the range of supply in the canal is considerable and blocks or small regulators have to be used to force the water under a system of turns, or during periods of low supply into the outlets.

In order to avoid confusion it may be as well to define " watercourse " as a small distributary furrow, ditch, or canal, which serves the smallest subdivision of land, or the water in which is under the control of and used by a single individual.

A " lateral " or " minor distributary " is a small branch canal taking off from a larger distributary or small canal which feeds a number of watercourses. A " major distributary " is a channel taking off from the main distributing canal or branch, which may supply either laterals or watercourses, or both, directly.

A watercourse must never carry more water than can be conveniently handled without waste by a single individual. This maximum discharge will vary according to the contour of the country, the method of irrigation practised, and the skill of the irrigator, but should never exceed three cusecs, and is generally about one cusec.

A lateral or minor distributary frequently has to be worked in sections, the water being headed up at intervals to feed one or more watercourses. If the lateral is provided with properly constructed regulators or " stops " for holding up the water at intervals, a discharge capacity up to about ten cusecs may be provided, but if temporary and primitive devices for turning the water into the watercourses are employed the maximum discharge capacity permissible will be very much less, namely, about three cusecs.

Major distributaries or small canals should have as few offtakes as possible, and if at all practicable regulators and other devices for heading up the water should be avoided; the existence of such works makes anything in the nature of a permanent regime impossible, as they encourage silting.

Whenever canals or major distributaries have a capacity in excess of about twenty cusecs direct outlets to watercourses should be avoided. These should be fed from the laterals only.

The main canal serving a particular area should, of course, be aligned on the main ridge, or along the high ground bordering the area. Major distributaries, minor distributaries, and water-

courses will be aligned along the secondary, tertiary, and minor ridges respectively. The distribution system must be built up from the smallest irrigation units, namely, the areas served by single watercourses, and the boundaries of these should be, in general, either natural drainage lines or ridges or the limits of command by the furrows. On no account must any main or secondary or in fact any defined natural drainage line be obstructed by any distributary or watercourse. When, owing to the peculiar topographical conditions, command of an area can only be obtained by carrying a distributary or lateral across a defined drainage line, such distributary must be either carried across in a flume, permitting unimpeded flow of the natural drainage underneath, or, alternately, it must be taken across in a siphon buried below the ground level. All main drainage lines should be maintained in a clean and unobstructed condition, and all cultivation should be stopped at a certain distance from the lowest point, the distance depending upon the maximum flow normally carried by the depression or drain.

All accidental waste and all seepage or storm water should flow naturally and directly into these drains. Where, as is sometimes the case with small holdings, a lot or holding is not contiguous to one of the defined drainage lines, it is necessary to cut an artificial drain in such a manner that no adjoining land of another owner is likely to be damaged by seepage or waste irrigation water from other properties. These artificial drains should be cut as part of the original scheme. Cutting drains across properties which have been occupied and developed is apt to cause trouble.

From the above it will be evident that irrigation boundaries are inclined to be irregular areas, as nature does not work in straight lines, squares, or rectangles. This important fact is very frequently overlooked by surveyors when laying out irrigation settlements. In South Africa it has been common in the past to survey the settlement lots regardless of irrigation requirements, and before any detailed consideration has been given to the reticulation. It has also been a common practice to make all the holdings exactly the same size and square or rectangular in shape. All this has led to much subsequent trouble, as it was bound to do.

In northern India, where are vast areas of extremely flat land with gradients of 1 in 4,000 and less, the square system of settlement has many advantages; and on the newer Punjab

canals several millions of acres have been surveyed into twenty-five acre squares, each square being again subdivided in twenty-five one acre squares. Under the favourable conditions there obtaining it is possible in the vast majority of cases to command each twenty-five square acre from one point on its boundaries, and also to lay out a system of subsidiary watercourses which will bring the water to the highest point on the margins of each one acre subdivision. In South Africa the areas which lend themselves to such a layout are extremely rare. Irrigable lands have generally a much steeper and more complicated contour, and squares of the size usually required, viz., from twenty to forty acres, could generally be made use of on a very limited scale only. The virtue of squares and rectangles is after all very small. They look better on a plan and give the land, when developed, a particularly orderly appearance. They are more simple to survey, require fewer beacons, and are very easy to locate. Considerable as some of these advantages are they do not compensate for a complicated and inefficient irrigation system.

It is most strongly recommended that in South Africa the engineer responsible for the general design and construction of the scheme should likewise be responsible for the layout of the holdings. These holdings will have to be finally surveyed by a licensed Government land surveyor, who alone is empowered by law to prepare the survey diagrams on which land titles are based, but the location of the boundaries of each lot should be fixed by the irrigation engineer.

In order to illustrate the main features of an irrigation distribution system an example is given of a somewhat typical piece of irrigable land adjoining one of the larger rivers in South Africa (Fig. 31).*

The irrigable land is located in a defined valley and is composed of alluvium. The margins of the irrigable lands are demarcated by the hills on either side of the valley, or by the escarpments of old natural terraces or benches. As a rule such valleys are of no great width. These valleys have generally a characteristic cross section. The alluvium immediately adjoining the river is high. This is due to the fact that during high floods in the past, when the troughs of the rivers were less deeply cut than is the case to-day, whenever the water overtopped the banks the heavy silt was dropped close to their edges. As one proceeds away from the river channel the silt

* At end of book.

tends to become finer and the surface of the ground will be found to slope away from the river banks towards the sides of the valley. The lowest point is generally found to be in a depression or creek, which is near the rising land on the margins of the valley, and, where the valley sides are precipitous, this depression will be frequently found immediately under these steep sides.

If the valley is of considerable width the contour may be less simple.

In the lower reaches of many large South African rivers the valleys in cross section show a series of great steps descending from each side down to the river, which meanders about in the lowest portion or platform of the valley. The steps are sometimes steep escarpments twenty or thirty feet in height. In other cases they have become worn down to an easy contour by erosion. Each terrace marks an epoch in the geological history of the valley.

Within the limits of the lowest level or terrace now occupied by the river channel the characteristic section above described will be generally found to apply.

Fig. 31 shows a portion of a comparatively narrow valley on one side only of a river. A ridge of high ground runs in close proximity to the river and falls away towards the valley margin to a drainage line running down the valley at no great distance from the rising ground bordering it.

In the portion shown there is only little irrigable land on the farther side of the marginal drainage, but it is assumed that in other portions not shown the valley opens out, and that there exists a sufficiently large area of irrigable land beyond the drainage line to justify the location of a distributary on that side.

The main canal is shown aligned on the ridge near the river, whilst a branch distributary, taking off from the main distributary higher up than shown, is carried along the side of the valley beyond the drainage line. This drainage line is a main irrigation boundary, and is crossed by no watercourses. The land is shown cut up into holdings of various sizes, ranging from fifteeen to thirty-five acres, but were it desired to do so they could all be made of uniform size.

Each holding is served from a single outlet in the distributary, and the main watercourse system is shown in each case. The arrows indicate the direction in which command is secured.

The watercourse systems shown do not, of course, embrace the entire system. In order to bring the water to the highest point of each quarter or half acre check a considerable number of subsidiary watercourses will be necessary. It will be seen that all natural drainage lines, both large and small, are kept free, and generally form the boundaries of holdings.

The important question of roads is often overlooked or given inadequate attention. Each holding must be linked up with the main road system by a road which can be maintained in a serviceable condition at all times. In the case of most irrigation settlements the roads are execrable and degenerate into badly kept drains. Such a state of affairs is not only inconvenient but imposes a serious burden on the farmer. A good service road should be maintained alongside the main canal, and on that side of it on which there is the largest extent of irrigable land. If there is a trunk road in the neighbourhood there should be connecting roads at reasonable intervals. Roads into the various holdings taken off from the canal road should be aligned to the fullest extent possible along the ridges and should never run along the drainages. They should, in fact, follow the main watercourses. In all cases a sufficient width of land must be given up to a road to maintain it, and this is especially necessary if road and watercourse run side by side. If holes are dug into the road to stop breaches in the watercourse, or to make " stops " in that channel, the road will very soon degenerate into a quagmire or drain.

A main road system is shown on the plan. The alignment of minor roads will depend upon the detailed layout of the land, but the same principles apply in these cases also.

Culverts must be made to pass all watercourses across the canal road. If a road on both sides of the canal is not justified, then bridges must be provided so as to give access to the holdings on the opposite side of the canal. As bridges, if properly constructed, are costly structures, their number should be reduced to a minimum by making short lengths of road on the opposite side of the canal so that small holdings will be served by a single bridge.

Drifts through canals, major and minor distributaries should under no circumstances be allowed.

In order to keep the larger canals, etc., secure from damage by cattle, etc., they should be fenced on both sides. When it becomes necessary to water stock in the canal, specially paved

drinking places must be provided, the canal and banks being securely fenced off on both sides.

Provision for Dealing with Seepage Water.—As there is considerable percolation from an unlined canal, there is often great danger of land contiguous to the canal, on the lower side thereof, being injured by too much seepage water, which gives rise to waterlogging or brak trouble. Wherever intensive irrigation is carried on on a large scale a complete system of artificial drainage of the irrigated area becomes necessary. In the first instance it is, however, advisable to construct drainage ditches on the lower side of the canal and above the adjoining irrigated lands. In most cases open ditches will suffice. In some cases it may be necessary to go to the expense of installing a sub-surface drainage system, using drainage tiles or French drains.

The water percolating into these drains must be led directly into the main natural drainages and removed from the irrigated area as quickly and as directly as possible.

Methods of Distribution.—Broadly speaking, there are four methods of distributing water to farmers under a canal system : firstly, by the continuous flow of water from the canal throughout the irrigating season ; secondly, by intermittent flow, the farmer obtaining a certain definite discharge over a definite period of time and at fixed intervals ; thirdly, intermittent flow, under which the farmer gets water whenever it happens to be available ; and fourthly, the delivery of water to a farmer on application.

With regard to continuous flow, it is essential that the volume of water delivered to each irrigator shall be of sufficient volume to enable him to water his land rapidly. It is impossible to irrigate even a small area of land with a trickle of water without causing excessive waste, as the proportion of water lost by evaporation and absorption is disproportionately large. Continuous irrigation, therefore, implies that the volume of water delivered to the irrigator throughout the irrigating season shall be a workable quantity—certainly not less than one-half a cusec, and preferably a larger amount. The method of continuous irrigation is only feasible where the individual irrigator possesses a sufficiently large area of land to enable him to take water continuously. With very small holdings which can be completely irrigated within a week or less with a supply say, of one cusec continuous flow is impracticable.

On a perennial canal system the most efficient method of distribution is the second class. This method involves the

distribution of water by turn, or under a system of continuous rotation, and it is the standard method adopted in all irrigating countries. During a turn the farmer receives the maximum volume of water which he is able to handle efficiently for a fixed period of time, generally calculated in hours. As soon as his turn comes to an end his outlet is closed, and he is debarred from taking any more water until his turn comes round again. To the farmer this system is very much more satisfactory than the one under which he has to take water continuously. The strain of water leading only lasts a comparatively short time, and until his turn comes round he is free to devote himself to other general work on the farm. This method generally leads to the most economical use of water.

The third method, namely, intermittent flow at irregular intervals, is applicable only in cases where the supply of water is intermittent and erratic, as, for instance, on flood water canals, which are so common in the Cape province of South Africa. In these cases the distribution system is designed to work more or less automatically, so that when a freshet comes down the river suddenly everything is ready for the water to flow on to the checks with a minimum of supervision.

The distribution by application is, of course, only possible when the source of supply is constant. There are many serious drawbacks to this method, the main one being that it is impossible at any time to forecast when the demand for water will be heavy, as during periods of great demand the amount of water to be delivered to the applicants is likely to be, in the aggregate, greater than the capacity of the furrow.

In general there is no attempt made to measure accurately the volume of water delivered to each irrigator. Outlets or "turnouts" are designed to divert a certain discharge, say one-half or one cusec, from the main furrow into the farm watercourse or ditch under an assumed normal head of water, and on the supposition that both the main furrow and the head reach of the watercourse are in good repair and free from silt deposit.

In actual practice the discharge will be found to vary very considerably, and in all cases when the demand for water is great and the farmer is not taking all the water in the furrow, but has to share it with others, constant attempts are made to divert a greater amount than that to which the farmer is entitled. The supply of water to individual holdings, whilst based upon a theoretically correct allotment, is really a very

rough approximation to the ideal aimed at. Distribution according to the methods ordinarily adopted is never conducive to economy in the use of water. To achieve this it is necessary to supply water by measurement and to charge for it by volume.

In some countries, such as Italy and to some extent in the United States, where irrigators are educated and experienced, distribution by volume is practised, but the appliances used are by no means ideal, require considerable supervision, and are rarely proof against interference with fraudulent intent.

The appliance required is one of two types: firstly, one which will deliver a fixed discharge within certain wide limits of working head in the feeding canal; and secondly, one which will deliver the required discharge when the distributary is carrying its normal full supply, and, when carrying more or less than this discharge, will deliver proportionately increased or decreased supplies.

Appliances of the first class are called modules. These were designed and used in southern Europe very long ago, the Spanish and Italian modules being classical examples. Many later varieties are modifications of these. In most cases, however, the working head required is greater than can be sacrificed in the average distributary, and for this reason alone the employment of these types has been impracticable.

The subject of modules was investigated on a very comprehensive scale in India, where the want of an efficient module is very acutely felt. The late Mr. R. G. Kennedy devoted much study to the problem, and under the instruction of the Punjab Government carried out a series of elaborate experiments during 1904 and 1905, and succeeded in designing three types of modules which gave very accurate results, with working heads of less than three inches, and which were, moreover, immune against interference. The Kennedy modules are fully described in Punjab Irrigation Paper No. 12 (Revised Issue), published in 1908. Kennedy outlet modules are manufactured by Messrs. Burn & Co., of Howrah, India.

Subsequently a very efficient and ingenious module was invented by Mr. A. Shirra Gibb, likewise a Punjab irrigation engineer. This module is now manufactured and sold by Messrs. Glenfield Kennedy, Ltd., of Kilmarnock. For irrigation watercourses the Gibb module is probably the best device yet produced, as it is not only efficient but simple, cheap, and contains no working parts.

The second class of appliance is represented by the Kennedy "Gauge Outlet," or rateable semi-module, which is an extremely simple and cheap device, and is likewise described in the Punjab Irrigation Paper referred to above.

Although the installation of modules or semi-modules on a very extensive scale may involve a considerable outlay of money, it is most desirable that the system of sale of water by volume and not upon an acreage basis should be adopted, more especially in countries like South Africa, where the amount of water available is very inadequate and where irrigation works are very costly.

Meters in the ordinary course are unsuitable, both on the score of expense, delicacy, and excessive working head required.

Whilst the outlets from the public furrow into the lateral or watercourse are always permanent structures consisting of pipes, concrete, brick or wooden sluices, controlled by shutters, valves, or similar devices for opening and closing them, insufficient attention is paid to these details on the farmers' watercourses, reliance often being placed upon the spade and some earth dug from alongside the irrigation channel. These primitive devices invariably lead to inefficiency and waste of water, as each spot where the water is habitually turned into a branch channel or check becomes surrounded by a number of deep pits from which earth has to be taken from time to time to make the "stops." After a short period of working it becomes impossible to keep the watercourse in a reasonably good state of repair.

Small "turnouts" formed by concrete blocks with grooves for little wooden or steel shutters should be made. These are inexpensive, make water leading simple, and obviate all the trouble and mess which is inevitable with the more primitive method.

The method of irrigating land, that is, the way in which the land is laid out in relation to the water furrows, largely determines the amount of water required to irrigate a given area. An efficient layout must not alone lend itself to the highest degree of economy in the use of water to produce a good crop, but must allow of the land being irrigated rapidly and with the least amount of labour. The particular method adopted depends entirely on the local conditions obtaining. Irrigation water may be supplied either below the ground, i.e. sub-surface irrigation, or above the ground, i.e. surface irrigation.

The former method is much the more economical in the use of water as heavy evaporation losses are minimized. The method is costly, however, and is only justified when high-grade crops are grown and water is very scarce. Though examples of this method exist in the United States, the area so irrigated is very limited and the system will not be dealt with further.

Surface irrigation is practically the universal method adopted throughout the world. This is effected in two different ways: firstly, by flooding the entire surface of the area to be irrigated; and secondly, turning the water into a number of furrows made with a plough or similar appliance. Flooding is the more common method.

Where the slope of the land is steep and the soil light and inclined to become eroded or guttered the furrow method is the more practicable and is generally economical. This method is particularly suited to the irrigation of orchards and for crops which are not mown or harvested by means of mechanical appliances. Surface flooded land is laid out into small areas known as checks and other local terms, and these are generally narrow in comparison to their length. Where the slope of the country is very small they may be laid out with their long axes more or less at right angles to the direction of the contours. When the ground is steep, however, or at all inclined to wash, the checks should either be aligned with a very small gradient and making a very small angle with the contours, or the land must be terraced; that is to say, the slope of the land must be broken into a series of benches or steps, each bench being partly in cutting and partly in embankment. There is a great variety of ways of laying out land for surface flooding, particulars of which will be found in most modern textbooks on irrigation practice or irrigation engineering. In South Africa irregular or steep country is almost always terraced, as much as twenty pounds per acre being sometimes spent on developing the land for irrigation.

To obtain the greatest economy in the use of water in irrigated land nothing is more important than the careful levelling of the land to be irrigated and its subdivision into small lots, generally known as "akkers" in South Africa and "checks" in the United States.

It is in these matters that the novice invariably goes wrong. Even if he succeeds in getting his land fairly well graded and levelled he invariably makes his checks much too large.

In northern India on all the State canals it is a regulation (non-compliance with which is a punishable offence) that the land to be irrigated shall be divided up into checks approximately an eighth of an acre in extent, and that each check shall be fed independently at its highest point from the adjoining watercourse.

In order to demonstrate to the cultivators in the Punjab the great advantage to be obtained by strict compliance with this regulation some elaborate experiments were undertaken annually at a large number of places between the years 1904 and 1910, the author being in general control of the experiments during the first two years. The method followed in every case was as follows: Two contiguous plots of land were selected, which were not only identical as regards soil, but whose history during the previous few years as regards cropping, fertilizing, etc., was as nearly as possible the same. Whilst both plots were farmed by the same person, being the owner, one of the plots in each case was carefully prepared for irrigation by the particular departmental engineer in charge, and was likewise irrigated at his discretion and under his immediate supervision, whereas the adjoining plot was prepared and irrigated by the owner without any interference. The same crops were grown on each of the two plots. The water delivered on to the land at each irrigation was carefully measured in the case of both plots, and the crop obtained was likewise carefully weighed in each case. The experiments were in the nature of competitions between the departmental engineer and the native farmer. Out of the large number of experiments carried out during seven years a considerable percentage had to be discarded owing to the results being spoilt by various causes, such as untimely or excessive rain, crop failures due to causes over which it was not possible to exercise control, or to mistakes on one side or other. The crop grown in every case was wheat.

In reviewing the final series of experiments the Hon. Mr. W. B. Gordon, C.I.E., Chief Engineer, Irrigation Works, Punjab, states that the results corroborate those of the experiments in previous years, namely, that the ordinary cultivator uses fully twenty per cent more water than would be required to give the same outturn if the water were applied with due regard to economy, especially in the matter of dividing the fields into compartments or checks of a suitable size and delivering the water into each check separately. The results show generally that the saving in water effected in the case of the departmentally

controlled plots was greatest in the case of the first watering. Cases were not uncommon where the cultivator used as much as fifty per cent more water than was necessary. The actual saving in the case of the Government plots was more than twenty per cent, but it could hardly be expected that when irrigation is practised on a large scale the same care would be devoted to these matters, as was the case of the experimental plots. There is always a certain amount of superfluous flooding which cannot be prevented, and the twenty per cent saving must therefore be considered an attainable result. In connexion with these experiments it must be remembered always that even when left to himself the Indian irrigator, who is highly skilled in the art, invariably divides up his land into checks, which in most other countries would be considered small. Mr. Gordon stated, in conclusion, that if water were used with due economy by the cultivator the irrigated area might be extended by twenty per cent, allowing for further losses by percolation, etc., in carrying the water farther afield.

It is often urged by farmers, who are comparatively new to irrigation farming, that the labour entailed in laying out irrigation lands into small plots is too expensive and water leading too complicated. This, however, is not true ; and furthermore, no country depending upon irrigation can afford to sacrifice large areas of potentially irrigable land in order to pander to scruples and wasteful methods of inexperienced farmers.

It must not be assumed that the elaborate layout above described is peculiar to India. Amongst the most elaborate and efficient layouts of irrigation lands which the author has seen are those to be found in southern and eastern Spain. In that country the water is scarce, and the cultivators, with centuries of experience behind them, have brought the economical application of water to the land to an extraordinarily high stage of efficiency. Checks half an acre in extent are comparatively rare. In most cases they are very much smaller. Thus in the Segura Valley many lands were found to be laid out in checks 50 feet long and 6 feet wide.

Where agricultural machinery is employed whilst the crop is on the land, such as cultivators, mowers, etc., very small checks are not practicable, but it is quite possible to lay out checks a quarter and even an eighth of an acre in extent by making them about 30 feet in width and 360 or 180 feet in length. A very small gradient should be given to the check in

the direction of its long axis. The dividing walls between such checks should be broad and low with an easy contour, so that agricultural implements can be readily taken from one check into another.

Where the land is laid out in a series of long and narrow checks the feeding watercourse should run along the top end of these checks so that the water can be turned into each separately. When irrigating a long check the water should not be allowed to reach the bottom end before it is cut off at the top. If this is done much waste and over-irrigation of the upper portions will result. According to the slope of the checks the water should usually be cut off when it has reached a point from two-thirds to three-quarters of the length. The water already contained in the check will then suffice to cover the remaining one-third or quarter of the area. When very long checks are used it is nearly always found that the crop in the upper portion is inferior to that in the lower portion. This is due to the fact that in order to get the water down to the lower end the upper portion has received an excessive amount of irrigation.

For efficient irrigation the proper levelling or grading of the land is essential. A naturally well-drained, fertile soil consists of an abundance of particles loosely held together by cementing and flocculating agents. A "flocculator" is a chemical substance which causes the microscopic colloidal particles of clay, which will remain almost indefinitely in suspension in water, to collect or coagulate into flocks or flakes. A deflocculator breaks up such flocks into colloidal particles again.

Arthur Stead, in a paper read before the South African Society of Civil Engineers on the "Agricultural Aspects of Irrigation" (Minutes of Proceedings of the S.A. Society of Civil Engineers, Vol. XVIII, 1920), deals very fully and lucidly with the effect of irrigation on the physical conditions of the soil, and most of what follows is extracted from his paper.

The action of irrigation water is, unfortunately, a powerful factor in breaking down the granulated structure of fertile soils. The deleterious action is least when the water contains salts which are flocculating agents; it is greatest when the water contains the strongly deflocculating salt sodium carbonate.

Soils most likely to be injured from this cause are loams and clays.

Artificial drainage produces a deep and well-developed granulation of the soil. This allows of the ready penetration

of water, and also, what is of the greatest importance in the case of fine grained soils, it provides a ready means of exit for the imprisoned air, which is the greatest obstacle to the easy penetration of water into a fine grained soil. It likewise allows for the quick removal of surplus water.

With the exception of very sandy soils all irrigated soils should be drained in order to get the best out of them and to maintain their fertility.

The action of water applied in irrigating land tends to produce the closest packing of the particles forming the surface soil, necessitating constant cultivation in order to counteract this tendency.

This compacting action of water becomes of great importance in laying out or grading lands for irrigation, especially when it is intended to grow perennial crops, such as lucerne.

Grading generally consists of loosening and scraping the soil from higher areas into depressions, or on to sloping ground lower down to form a level terrace. The fill is generally more loosely packed than the area which has been scraped. The consequence is that when areas thus treated are irrigated the compacting effect of the water is greater over the filled area than over the scraped area, and what was formerly a depression will again become a depression unless the effect of the compacting is made good by further grading.

Uneven land which is graded will invariably require further grading after water has had time to compact the looser portions in the fill. If such further grading is not undertaken the land will become uneven. Depressions will form in which water will accumulate and spoil the crop, and high places may be difficult or impossible to irrigate. Should sodium carbonate concentrate below such depressions serious deterioration of the soil may take place.

It is obvious, therefore, that extreme care must be taken when grading land; and as it is impossible in most cases to achieve finality after the initial grading, the establishment of perennial crops, such as lucerne, should be avoided on newly graded land. It is much better to grow annual crops, such as cereals, for a year or two until the grading has proved to be satisfactory and permanent.

Bad grading means great waste of water, poor crops, and impoverishment of the soil, and the irrigation engineer should pay as much attention to the preparation of land for irrigation by the farmer as he does to scientific distribution.

CHAPTER X

IRRIGATION AND BRAK

Brak, or alkali, as it is termed in America, is a curse to the irrigation farmer in arid and semi-arid countries the world over, and has been the cause of much trouble in the western United States, India, Egypt, as well as in South Africa. The term " alkali lands " is used somewhat loosely, and in general denotes that the soluble salts contained in the soil have become concentrated, through evaporation and capillarity, to a sufficient degree to become visible on the surface or injurious to vegetation. There are a number of salts which may become harmful, but the three worst offenders are the carbonate, the sulphate, and chloride of sodium.

Vast areas of the globe in the regions of low rainfall north and south of the tropics are brak lands in varying degree in their natural state, and present generally a dreary and depressing appearance, being often devoid of all vegetation and covered with large patches of white efflorescence of the sulphate and chloride of sodium. Where vegetation exists it is largely useless and repellent. The vegetation of brak soils may be very valuable for grazing, as is the case in the " Karroo " of South Africa with " ganna " and " slaai " (mesembryanthemum varieties), and the salt bushes of Australia and portions of North and South America. In other cases the vegetation is quite useless for any purpose.

The most harmful salt is sodium carbonate, known as " black brak " in South Africa, or " black alkali " in America. This name is due to the fact that it dissolves certain organic soil constituents, forming a blackish solution, and the ground has the appearance of being stained in spots or patches.

In the case of arid lands the rate at which soluble salts are accumulated, due to the constant disintegration of the soil particles by physical and chemical means, is greater than can

be leached out and disposed of by the scanty rainfall and natural drainage of the country. The solutions of these salts tend to become supersaturated when brought to the surface by capillarity and there precipitated. The more insoluble salts cannot form sufficiently concentrated solutions to become harmful, and therefore do not usually constitute a brak condition.

The rains in these countries may cause those concentrations to move in the direction of the natural drainage, and when heavy may cause them to sink into the soil; but the long periods of hot, dry weather invariably cause them to rise again by capillarity. As the rainfall increases in quantity or regularity so does its capacity to deal with any excessive concentration of alkali, and the evil is reduced or entirely absent.

In land strongly impregnated with these salts the seeds of crop plants germinate with difficulty, if at all, and the plants after a short sickly existence shrivel up and die before coming to maturity. In soils moderately impregnated with brak the plants are dwarfed and produce but little fruit or seed.

Experience in the western United States shows that in the case of sodium carbonate a concentration of an average of one-tenth of one per cent of this salt in the layer of soil supporting the root system renders the cultivation of most useful crops impossible. In the case of the chlorides and sulphates a somewhat larger degree of concentration can be tolerated.

Although there is by no means a consensus of opinion as to what percentages of brak in the soil are tolerated by crops—and this is not to be wondered at considering the varying conditions under which crops are grown—the following rough classification appears to have been adopted by the United States Bureau of Soils ("Soils: Properties and Management," Lyon, Fippin & Buckman, New York, 1919, page 396):

Percentage of Total Salts	Percentage of Black Brak	Crops which can be Grown
0·00 to 0·20	Less than 0·05	All crops grow
0·20 to 0·40	0·05 to 0·10	All but the most sensitive
0·40 to 0·60	0·10 to 0·20	Old alfalfa, sugar beet, barley and sorghum
0·60 to 1·00	0·20 to 0·30	Only most resistant plants
1·00 to 3·00	0·30 and above	No plants

Different plants vary greatly in their powers of resistance to brak, and from investigations made in the United States it has been possible to arrange a long list of common crops in the order of their resisting powers, from which the following selection is made: (1) Salt bushes, certain Karroo mesembryanthemum species, etc.; (2) Sorghum; (3) Sugar beets; (4) Kaffir corn; (5) Lucerne (old); (6) Barley; (7) Beans; (8) Vines; (9) Wheat; (10) Oats; (11) Lucerne (young); (12) Potatoes; (13) Onions; (14) Peach; (15) Apple; (16) Apricot; (17) Citrus fruits.

This list is of importance, and much disappointment and loss can be prevented by avoiding the growing of crops and trees sensitive to brak on soils in which the concentration of soluble salts is somewhat high.

If the land is slightly brak in its natural state it is a comparatively simple matter to make it worthless in a very short time by faulty irrigation works or mode of applying water.

An irrigation system may cause brak troubles in various ways, of which the following are the most common: Seepage from irrigation canals; over-irrigation; the use of water for irrigation which has become charged with an excessive amount of salts, either by too long a period of storage, or if the water is the drainage from brak lands. Leaky or porous canals and ditches are one of the most common sources of trouble. The seepage from these furrows gradually fills up the subsoil, which becomes saturated so far as it is permeable, and the water table rises to within capillary reach of the soil surface, and even reaches the furrow level, thereby causing local waterlogging, drowning the root systems in the vicinity, and bringing to the surface much of the brak salts normally diffused throughout the depth of the soil. It is a most common occurrence to see the surface of the land on either side of a canal, or, at any rate, on the lower side, covered with an incrustration of brak, and the crops grown within a considerable distance from the furrow become poor and eventually the land becomes unfit for cultivation.

The only method of preventing this is to make the furrows watertight by lining them, and by intercepting as much as possible the seepage water by means of deep lateral drains.

Over-irrigation, by which is meant the application of more water to the land than is required by the crop, is equally damaging to the soil. When more than five inches of water is applied there is usually a considerable loss from seepage. This loss,

apart from being very wasteful, causes a rapid rise in the subsoil water and the consequent rise of brak salts, and it likewise causes the concentrated solution of brak to flow towards the lower levels, causing those lands to become poisoned with an excess of those salts.

To prevent these evils two things are essential: firstly, water for irrigation must be used in the most economical manner possible; and secondly, irrigated land which is liable to brak troubles must be covered by a comprehensive and properly designed drainage system, which will cause all seepage water charged with injurious salts to be carried away as rapidly as possible to the natural and unobstructed drainage lines of the country. The use of water for irrigation which contains an excessive amount of salts is most dangerous, and could only be practised if the ordinary irrigation rule is reversed and the saline water is used in such excessive quantities that thorough drainage is assured. This, however, is only possible with a sandy loam or other porous soil. Irrigators should not be tempted to draw upon supplies which have become too concentrated to use with safety.

The use of saline water is inadmissible. If the water contains a very small percentage of sodium carbonate it cannot be used on average soil without becoming a source of danger, as it adversely affects the texture of the soil.

For such water to be used with safety it must also contain flocculating salts in abundance, or, alternately, the soil must contain a store of such salts. If this is not the case, the cementing action of the sodium carbonate causes the formation of a particularly bad type of hard pan. In the Karroo districts of South Africa, where the water is diverted from intermittently flowing rivers, the water is frequently unsafe to use for irrigation when the flow has almost ceased, and brak salts derived from seepage, which were temporarily held back by the rise of the flood water, are released again. Turbid water is generally safe to use, but whilst clarification of the water may be caused by an injurious coagulent, like sodium carbonate, it may also be due to calcium and magnesium bicarbonates, which are harmless. The latter salts are mainly responsible for the clarification of the water in the Brak, Fish, and Sundays rivers in the Cape province.

In the drier parts of South Africa surface waters, which are impounded in a reservoir, frequently contain brak salts, and if

such waters are stored for too long a period the concentration of these salts may become excessive and the water will be unfit for irrigation purposes.

Dams in the Karroo are frequently built on the line of a dolerite dyke, and such dykes may impound a large volume of underground water, which may be brak or fresh. In the former case, if water in the reservoir is impounded for a lengthy period, such an underground supply establishes direct connexion with the water in the reservoir, and the latter will then become contaminated by connexion currents and diffusion.

Evaporation will not necessarily produce a harmful concentration of brak unless one or other of the above-mentioned conditions obtain.

The worst case in South Africa of which any record exists is that of the Van Wijks Vlei reservoir in the north-western districts of Cape province, which is an extremely arid part of the country, the mean rainfall being only six and a half inches, and the catchment area is highly impregnated with brak salts. Some information regarding this matter is given by A. D. Lewis in a paper read before the South African Society of Civil Engineers on the " Run-off and Reservoir Losses at Van Wijks Vlei in 1915 " (Vol. XIII of the " Proceedings "). Quoting from this paper it appears that the average rate at which salts enter the reservoir is about 10 grains per gallon, and the concentration becomes dangerous for the lands below when it exceeds 80 grains per gallon. It therefore becomes necessary to use the water at a sufficiently rapid rate to prevent the concentration exceeding eight times its normal state at the time of inflow.

At times the concentration has been very high, exceeding 1,000 grains per gallon in 1903 with 2·0 feet of water in the reservoir, and 500 grains per gallon in 1908 with about 4 feet of water.

At Van Wijks Vlei experience has shown that the water is generally quite unfit for use if stored for longer than two years, provided a considerable volume of water is in the reservoir. During periods of severe drought the water becomes unfit for use much more rapidly. In order to avoid such harmful concentration it is undesirable in such cases to construct reservoirs of such capacity that surplus water will only be discharged over the waste weir at very rare intervals. Whilst spillways at full supply level should always be provided to deal with floods of exceptional magnitude, reservoirs of this type should, in the

author's opinion, be provided with means of discharging large volumes of water at comparatively low levels through large scouring sluices. By such means, it is held, the mass of water in the reservoir can be maintained in a fresher condition. This matter has also been alluded to in the chapter on conservation.

Whilst it is possible, with reasonable precautions, to avoid the deterioration of otherwise good land by irrigation, the problem of applying remedial measures to land badly impregnated with brak is a difficult and costly process.

Two methods have been adopted with success, but both involve the use of large quantities of water, which can, in most cases, be spared with difficulty, as the volume of water so required could be used to irrigate a larger area than that which it is desired to reclaim.

In South Africa it has, however, been ruled in an important judgment of the Water Court that subject to certain conditions the use of water of a public stream for the purpose of reclaiming brak land and making it fit for the production of crops under irrigation is a legitimate use, and could not be held to be an unreasonable use of water.

No complete reclamation is possible without a complete system of under-drainage. As very few soils have sufficient natural under-drainage, the degree of success ultimately attained will depend largely upon the efficiency of the artificial drainage system, and the degree to which this can be carried out is dependent on the amount of money which it is economically feasible to spend on such works.

If under-drainage alone is to be relied on it becomes necessary to give very heavy applications of water to the land until the seepage through the soil has leached out all the injurious concentration of salts. For this purpose the land should be divided up into beds of not more than half an acre in extent, so that the whole area can be frequently and heavily flooded without undue waste of water. The intervals between successive floodings should not be long enough to allow the soil to dry out, otherwise a large percentage of the salts will be brought to the surface again by capillarity.

If possible it is best to have a crop on the land which is being reclaimed. Wheat is a suitable crop for this purpose as it stands flooding well, but no yield of grain must be expected. Its main value would be to shade the land.

Another method, which, however, should be supplementary

to under-drainage, is to divide the land to be reclaimed into small, level beds. These beds should be kept well flooded till the water impounded is a concentrated solution of brak salts. It should then be drained off and flooded again and the process repeated. This method is followed in the concentration of brine in salt pans for the production of common salt. It has been used successfully in India for the reclamation of land impregnated with sodium sulphate. This method is better suited to soils of the heavier types. Soils whose physical condition is likely to be damaged by such treatment should not be reclaimed by this method.

Many cases are on record where land heavily charged with the worst forms of brak has been reclaimed by under-drainage with tiles in the course of less than a year and returned to permanent fertility.

Where—as in parts of South Africa—basin irrigation is practised, great care must be taken not to allow a big depth of water to stand in the basins for too long a period, otherwise the land in the basins will become useless in a very short space of time. The basins should be of moderate size, with not more than three feet of water standing against the banks at the lower end, and the water should not remain impounded in the basins for a longer period than a week. In most cases forty-eight to sixty hours is sufficient to secure adequate saturation of unspoiled land in order to secure a crop of cereals.

Where the brak present is mainly sodium carbonate an improvement can be effected by dosing the land with gypsum. The latter must be well harrowed in or otherwise incorporated with the soil. The effect of gypsum on the sodium carbonate is to produce calcium carbonate, which is very slightly soluble in water and harmless, and sodium sulphate, which is a brak salt much less harmful than the carbonate.

For the prevention of brak troubles on irrigated lands the proper layout and grading of the land is of the greatest importance. This matter was fully dealt with at the end of the last chapter.

CHAPTER XI

IRRIGATION LAW

IRRIGATION development in a country is, to a large extent, dependent upon the law and practice relating to the title to and use of water in streams. In most of the newer countries, which have grown out of European colonies, in which irrigation is necessary, development has been much hampered by legal systems which were taken over from the mother countries, in which irrigation was either unknown or carried on on a very small scale only.

This has been markedly the case in both the United States of America and in South Africa.

In both countries the parent law embodied the doctrine of riparian ownership, which vests in the owner of riparian land the right that the flow of the stream on which his lands abut shall be maintained in perpetuity both as regards quantity and quality. The common law of England and of Holland was developed in humid climates, and the use of water in public streams was confined to driving water wheels, wool washing, navigation, domestic use, and the like. The law was, however, antagonistic to the consumption of water for growing crops under irrigation.

Irrigation development therefore in the Colonies led rapidly to radical departure from the original narrow common law principles. The right to divert and use water for irrigation had to be recognized in order to make the development of hot and arid regions possible. There has therefore been a gradual evolution of the law relating to water in streams, from the original riparian doctrine to what is termed in the United States the " Doctrine of Appropriation."

In a dry country, depending very largely for its agricultural development upon irrigation, the really efficient use of the water resources of the country is only possible under a system of

national ownership in all the water flowing in streams. An example of the far-reaching effect of such a system is to be seen in Upper India, where with State ownership of the stream-flow and theoretical State ownership of all land, and an enormous system of State irrigation, it has been possible to make the fullest use of the water available in the large perennial rivers, regardless of whether the land to be irrigated is riparian, or even within the catchment of the river from which the water is diverted.

Such an ideal state of affairs is impossible in South Africa, and in most other countries peopled by Europeans, unless popular ideas regarding individual ownership undergo a very drastic change.

The problem of each country is therefore to gradually develop and broaden its laws relating to the use of waters for irrigation and water power, so as to meet the requirements of the period.

In many respects South Africa has made very great strides with the development of its water laws, and has thrown off many of the shackles of the old parent legal system brought over from Holland, but for the fullest development of the very inadequate water resources of this vast dry country the present law cannot be accepted as the last word.

As each stage in the evolution of a national water law generally involves interference with the vested rights of land-owners—the most conservative element in any community and very jealous of their rights—progress is naturally somewhat slow.

The law relating to the use of water in streams was, until comparatively recently, based upon the common law of the various South African States, the foundations of which are in every case Roman-Dutch law.

In course of time an elaborate system of "Case Law" had been evolved, more especially in the Colony of the Cape of Good Hope, and this served to meet the simple requirements of the country until the close of the last century. After the Boer War irrigation development began to go ahead in the Cape Colony and Transvaal, and the old common law was found to be wholly inadequate to meet the peculiar requirements of the country. A comprehensive law relating to the use of water in streams was passed in the Cape Colony in 1906, and the Transvaal followed suit in 1908. At the time of union irrigation development was making such rapid strides that it was realized that

neither of these Acts would satisfy the requirements of the near future. As it was essential that there should be a uniform water law in all the provinces of the Union, advantage was taken of the occasion to enact a consolidated national water law, the provisions of which went as far as possible towards encouraging and facilitating irrigation development without departing further than was necessary from the old-established common law principles, and without interfering unduly with vested rights.

In a country such as South Africa, where the entire future agricultural development and prosperity is closely associated with the fullest use of the water which falls on the surface in the form of rain, or of water which is stored beneath the surface, the only way of effecting this in the most efficient manner possible would be by nationalizing all the available sources of water supply.

In South Africa, Rhodesia has succeeded in establishing a modified system of national ownership, which may be of great value to that country in the future.

The Union is not so fortunate. The law, as it now stands, is embodied in the Irrigation and Conservation of Waters Act, No. 8, of 1912, and its main provisions will be briefly dealt with. This law must be read together with its regulations, which are of great importance. One section of those, known as the Parliamentary Regulations, is as important as the law itself.

Surface water in the Union is divided into two classes—private water and public water. The definition of private water is unfortunately very difficult, and as it has been the subject of much controversy amongst lawyers, great caution must be exercised in dealing with the matter.

It is a principle of common law and of the Irrigation Act that every person is entitled to the exclusive and unlimited use and enjoyment of water rising on his own land, or of all water which falls or naturally drains on to the surface of the land, so long as it remains on such land and does not join or form part of a public stream.

A public stream is defined as a natural stream of water, which, when it flows, flows in a known and defined channel (whether or not such channel is dry during any period) if the water thereof is capable of being applied to the common use of riparian owners for the purpose of irrigation.

A stream may be in part private and in part public.

With private water and private streams the law does not concern itself, and it is often very difficult to decide whether water is really private or not.

The definition of a stream or whether any water is private or public rests with the courts.

The irrigation engineer being primarily concerned with public streams and public water attention will be confined to these.

There is no right of property or absolute ownership in any public water. Certain privileged persons termed riparian owners have, with strict limitations, the right to use such water on particular land, which is called riparian land.

Riparian land is primarily all land which constituted an original farm or grant, certificate of title, etc., through which or along any boundary of which a public stream flows. Also any portion or subdivision of such land unless it was deprived of its riparian rights when so subdivided. Crown or public land which has not been demarcated into farms is riparian in so far as it is contiguous to a public stream. A riparian owner is the registered owner for the time being of riparian land.

The law recognizes three classes of use of water in a public stream by a riparian owner. These classes in order of precedence are:

(1) Primary use; water necessary for domestic purposes and for the support of animal life.
(2) Secondary use comprises all irrigation requirements.
(3) Tertiary use comprises all requirements for water power, and all kinds of industrial uses.

Secondary use is subject to primary requirements being satisfied, and tertiary use is subject to both primary and secondary requirements being satisfied.

The law differentiates between two classes of water flowing in a public stream, viz., normal flow and surplus water.

The normal flow of a public stream is essentially perennial flow. A stream which contains no steady flow derived from springs, seepage, drainage from large swamps, etc., can have no normal flow. On the other hand, the legal normal flow in any stream is not necessarily restricted to such perennial flow. Provided that a stream carries an appreciable, even if small, discharge of the character described above, the normal flow of the stream includes such quantity of water which can be beneficially used for irrigation by direct methods, that is to say, by

direct diversion from the river on to the land to be irrigated either by means of furrows or pumps. It does *not* include any water which can only be beneficially used for irrigation if first conserved in reservoirs.

All water flowing in the river, at any point, in excess of the normal flow as determined by a competent court, at that point is termed surplus water.

What constitutes normal flow can best be illustrated by a couple of examples.

The Water Court was called upon to determine the normal flow in the Vaal River at the site of the large barrage constructed by the Rand Water Board. The Vaal is a truly perennial river and thus has a normal flow. Its discharge varies enormously, however. During the spring months it frequently falls very low, the discharge amounting to only a few cusecs. Its flood discharge is, on the other hand, very great and frequently exceeds 50,000 cusecs.

In the case of land riparian to the Vaal River, crops can only be grown under irrigation if water is available during August, September, and October, and unless the waters of the flood discharge are conserved so that they are available during the critical months the area irrigable is very limited. As the normal flow is confined to that discharge which can be used beneficially by means of direct irrigation only, this flow must be limited to the average discharge flowing during the critical months mentioned above. The court therefore determined the normal flow at the point where the Vaal River entered the storage area to be 250 cusecs, which was a very liberal interpretation, seeing that mean dry weather flow calculated over a period of ten years preceding the investigation was only 167 cusecs.

Everything over 250 cusecs is, in this portion of the Vaal River, therefore surplus water within the meaning of the law. In the case of a river which is fed by a permanent spring the normal flow would probably be the maximum yield of the spring or springs. Now, so far as the normal flow is concerned its use by riparian owners is much circumscribed, and the provision of the present statute law follows closely the old common law principles as developed in the courts prior to the passing of the Cape Irrigation Act of 1906.

Every riparian owner is subject to all existing rights of other persons entitled to the reasonable use of the normal flow of the public stream to which his land is riparian.

What constitutes reasonable use depends upon circumstances and is a question of degree. The primary requirements of other owners must be met. No more than a reasonable proportion of the water must be used by an owner for irrigation, consistently with similar rights of irrigation possessed or enjoyed by lower riparian owners. The water abstracted from the stream must be returned to the same stream lower down with no other loss than has been caused by efficient irrigation with efficient works; that is to say, the methods of irrigation employed must be good and economical, and the works must cause the least possible amount of loss.

The reasonable use of water depends, *inter alia*, upon the character of the river and the nature of the land, the total extent of riparian land belonging to all the owners concerned, the extent of frontage of each property to the stream, the distance which water must be carried in each case to reach the irrigable land, methods of cultivation locally practised, etc.

The apportionment of the normal flow amongst a number of riparian owners is a most difficult and troublesome matter. It can only be done with binding effect by the Water Court, and is generally a lengthy and costly process.

Wherever irrigation development has reached the stage where the available supply during the critical season is less than the demand a definition of rights by the Water Court becomes inevitable.

The position with surplus water is very different. Prior to 1906 every stream which did not flow continuously for more than six months in the year was a private stream, and riparian owners could do what they pleased with the water when available. From 1906 onwards restrictions have been placed upon the use of water flowing in such streams, which are now classed as "public." The surplus water in a public stream is, however, available to riparian owners for the use for irrigation of riparian lands without restrictions, provided it is not used *unreasonably*—by which is meant that the use must be of a beneficial character from an agricultural point of view.

Subject to the above restriction and to certain others which will be dealt with presently an upper owner may store or divert the whole of the surplus flow from a public stream without allowing any of it to flow down to lower owners. So far as surplus water is concerned, the policy underlying the Irrigation Act is to encourage the fullest use thereof, and more especially

by means of conservation works to the fullest extent possible. It is obvious, however, that the great freedom which is given to any riparian owner in the use of surplus water exposes a lower riparian owner to a very considerable risk if he carries out large and costly irrigation or storage works, which may be rendered useless if an upper owner, following his example, constructs similar and possibly even larger works, and thereby deprives him of all water. The Irrigation Act provides for this contingency and enables an enterprising owner, who has constructed or who desires to construct large works for the utilization of surplus water, to obtain protection for such works.

Such protection is obtained by making application to the Water Court. When doing so the applicant must submit full particulars as to the design of the works, how much water will be impounded or diverted, and the use to be made of the water on riparian land.

Every other riparian owner, both up- and downstream, who, in the opinion of the court, is likely to be affected by the granting of such protection, must be served with a notice by the applicant, setting forth all the details as submitted with the application. Every owner upon whom the notice has been served must, within six months of the issue thereof, declare whether he proposes to construct any storage or diversion works within the same catchment, details of which must be submitted to the court, within the next five, and in special cases the next ten, years. At the expiration of the period of six months allowed the Water Court then fully investigates the matter and determines the capacity of the storage work or diversion work for each of the schemes of the applicant and declarants respectively, and also the proportionate amounts of water which successive owners of the works under consideration shall pass to similar works lower downstream. The leading engineering features of each scheme are determined by the court, and, subject to the right of appeal to a superior court, the order of the Water Court is final. This therefore debars any other riparian owner from constructing works for the utilization of surplus water within the same catchment except with the permission of the Water Court. Such permission for works applied for subsequently may only be granted if the schemes originally protected are not prejudicially affected to any material extent. The policy of the legislation regarding protection is to encourage and protect immediate enterprise. An owner may be possessed of considerable

potentialities in the way of storage or irrigation, but his failure to undertake to carry out a scheme within the period allowed, whether due to lack of enterprise or to circumstances over which he has no control, is not allowed to hamper the enterprise of other owners in the same catchment.

It will be seen also that immediately an application for protection is dealt with by a Water Court all the original freedom attached to the use of the surplus water at once ceases, and this water is apportioned in a very rigid manner, not necessarily amongst all riparian owners, but possibly only amongst a small percentage of them.

No large scheme for the utilization of surplus water should on any account be undertaken without first obtaining protection therefor, unless such a scheme includes all the owners having a right to use the water, or all those who may be able to make use thereof.

The relation between the rights on a main stream and its tributaries are of importance.

With regard to the normal flow of a tributary the owners along it have a preference over the owners on the main stream.

Before surplus water of the tributary can be considered, however, a further certain quantity of water must be allowed to flow down so as to satisfy the reasonable requirements of owners on the main stream.

There may thus be a certain quantity of water in a tributary which is in excess of normal flow but does not constitute surplus water.

In the absence of protection owners on a tributary have full freedom of use of surplus water. Under a protection scheme, however, the main stream and its tributaries have to be considered as one.

A difficulty here arises as to the definition of the words "tributary" and "main stream."

The matter has been very ably argued by Mr. (now Sir) J. E. R. de Villiers, Judge President of the Supreme Court of the Orange Free State ("South African Law Journal" for November, 1921, Vol. XXXVIII, Part IV), and he holds that the tributary is a stream which joins another stream under such circumstances that the latter is the main stream both above and below the confluence. He differentiates between tributaries and branches of a stream which are all of equal status. Where main branches of the river are concerned riparian owners below the confluence of

two branches are likewise riparian to the portion of the stream above the confluence.

The above are the main provisions of the law regarding the rights to use the normal flow or the surplus water of a public stream. There are numerous complications concerning existing rights, prescription, the unreasonable use of water, and other matters which need not be gone into here.

The law makes provision for the use of the water of a public stream on non-riparian land within or without the catchment, but only in cases when it can be clearly proved to the Water Court that the water so diverted and used is not and can not be used now or in future on riparian land.

The Irrigation Act contains many provisions to encourage and facilitate the construction of irrigation works.

Thus, if an owner is not able to lead water on to his riparian land by means of works constructed within the limits of his own property he can acquire servitudes:

(*a*) To lead water across the land belonging to another owner. This is called a servitude of aqueduct.

(*b*) To abut a weir or dam on to land belonging to another owner. This is called a servitude of abutment.

(*c*) To submerge the land owned by others with water caused to be held up by a weir or a storage dam. This is called a servitude of storage.

If such servitudes cannot be arranged amicably they can be claimed through a Water Court. The compensation payable in these cases is limited to the average market value of the land actually occupied by the works or reservoir, together with an annuity or the capitalized value thereof for actual loss or inconvenience likely to be suffered by the exercise of the servitude.

The above-mentioned facilities as regards the acquisition of servitudes are essential to the development of irrigation.

Apart from the law regulating the use of water in public streams, etc., and to the acquisition of servitudes, a large section of the Irrigation Act concerns itself with executive and administrative matters the object of which are to promote irrigation development.

A State Department of Irrigation has been created under this law, which is endowed with very wide powers, and the main functions of which is to undertake the difficult and costly work of bringing to light and working up the irrigation and water

power possibilities of the country; formal propaganda; the promotion of promising schemes, either by State agency or by groups of riparian owners or individuals; the financing of irrigation schemes and their control.

Much very valuable work is done by the Department through its Hydrographic, Reconnaissance, and Meteorological Branches.

For various reasons, which need not be entered into here, the undertaking and working of large irrigation schemes by Government agency has in the past not been very successful, and during the past sixteen years most of the development has been effected by means of co-operative schemes promoted financed, and controlled by the State.

These co-operative bodies are called "Irrigation Boards." The provisions of the law regarding co-operative irrigation schemes are as follows: If three or more riparian owners are desirous that a particular irrigation scheme involving a number of owners be carried out they may present a petition to the minister in charge of irrigation matters, describing the nature of the scheme and praying that a certain specified area, being riparian land, shall be constituted an "Irrigation District." In practically all cases the scheme in question is one which has been worked up and recommended by the Irrigation Department. If the minister is satisfied that the persons signing the petition are owners of not less than one-tenth part of the area proposed to be irrigated, and generally of the *bona fides* of the petition, the contents thereof are published by the Government, and in due course a statutory public inquiry is held into the matter by the Director of Irrigation, or on his behalf. If, as the result of this inquiry, the minister is satisfied that the owners of not less than two-thirds of the total extent of the irrigable areas in the proposed district are in favour of creating an Irrigation District, such may be proclaimed by the Governor-General in Council, the boundaries of the district being either as set forth in the petition or as modified by the minister as the result of the inquiry.

It is thus possible for owners of two-thirds of the proposed irrigable area to force the proclamation of a "district" against the wishes of the owners of the remaining third of the area. After constitution of the district a voters' roll is prepared, in which every owner has one vote in respect of each five morgen or part thereof held by him, but no person can give more than ten votes. The voting power is thus in favour of the small

holder. An owner of 50 morgen and one of 500 morgen have each ten votes.

A Board, called an "Irrigation Board," is then elected, consisting either of three, six, or nine members, according to the discretion of the minister. These members are normally elected for a period of three years, and retire in groups of three by rotation. When the Board has been elected the subsequent development of the scheme is vested in this body.

The main functions of an irrigation board are to construct irrigation works, and subsequently to work and maintain them, and in order to finance such undertakings boards are entitled to levy rates (called "irrigation rates") on and in respect of every piece of land irrigable by the scheme or schemes controlled by the board. These rates are normally assessed annually and at a uniform rate, though differential rating is allowed in particular circumstances.

An irrigation board may, subject to certain formalities being complied with, and with the consent of the minister, raise, by means of loan, any money required by it for carrying out works or any of its functions. Although money may be raised from other sources the loans are, in the vast majority of cases, received from the Government as irrigation loans under the Irrigation Act.

Such Government loans are granted to irrigation boards in respect of approved schemes on the security of the irrigation rates leviable by the boards, and in such cases these rates are a first charge upon the land, with priority over all other charges. No transfers of land can be effected whilst irrigation rates remain unpaid, and they rank preferent to all mortgages on the property.

By far the greater portion of all irrigation development in South Africa has been effected by such irrigation boards, 87 of which were constituted up to the end of March, 1921. In 53 cases irrigation loans had been granted to boards aggregating nearly £3,400,000 in respect of works estimated to cost a total of £4,743,000. The total rateable area on the same date of these 53 boards amounted to 137,808 morgen, or 292,153 acres.

Reference has been made above to the Water Court. This is a special judicial court, created originally by the Cape of Good Hope Irrigation Act of 1906 to deal with all matters arising out of that Act. In subsequent Irrigation Acts these Water Courts have been developed and now possess very

considerable powers. The main idea underlying these courts is firstly, that they are composed of a judge sitting as president of the court, together with two assessors, who are otherwise on an equality with the judge. One of these is an engineer of the Irrigation Department, nominated by the minister, and the other is a lay assessor drawn by lot from panel of landowners of standing and repute. Secondly, that a court is constituted afresh in respect of each application, and the lay assessor is, in every case, a man possessing local knowledge, but in no manner personally interested. Thirdly, the court sits as close as possible to the spot with which the application or dispute is concerned.

The presiding judges are termed "Itinerant Water Court Judges," and together with their registrars do no other class of judicial work. They thus become expert water judges. The departmental engineer brings his expert technical knowledge to bear on the subject, whilst the lay assessor is frequently of considerable value owing to his knowledge of local conditions, more especially the local agricultural soil and climatic peculiarities.

These courts are peripatetic and frequently hold sittings at several places in the course of hearing a particular case, apart from making extensive tours of inspection.

An appeal lies from the Water Courts to the Supreme Court and thence to the Appellate Division, or, by consent of all parties, the appeal may be taken direct to the Appellate Division.

The country is divided up into a number of Water Court Districts, which are essentially hydrographic units, and each district has its own panel of lay assessors.

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